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AMERICAN SOCIETY

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CIVIL ENGINEERS.

(INSTITUTED 1852.)

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No. 898.

HEAVY RAILWAY CONSTRUCTION IN WYOMING.

By J. I. Boggs, Assoc. M. Am. Soc. C. E.

Presented March 20th, 1901.

WITH DISCUSSION.

There are many in the East and South who have only faint ideas regarding the heavy construction being generally carried on by the great transcontinental railway lines in their efforts to reduce grades and straighten alignments; and these slight ideas are usually gathered from brief and incomplete notices appearing at irregular intervals in the daily and technical press.

Among others, the Union Pacific Railroad has shown remarkable activity in this direction. Its line has been shortened by many miles, and its grades reduced to a maximum of 0.82 of 1%, compensated 0.03 for each degree of curvature, the maximum curvature allowed being 4 degrees. On all new construction everything is being carried out to a perfect working model.

The highest summit on this line is at Sherman Hill, an elevation of 8 247 ft. above sea level, situated 33 miles west of Cheyenne, and 23 miles east of Laramie, Wyo. This has always been the *bête noir* of the transportation department. To overcome some of its difficulties a

corps of engineers was placed in the field early in 1899 to locate a line from Laramie to Buford, the latter being a small station 7 miles east of Sherman. Several months were spent in locating the line, the final result exhibiting a large amount of curvature and an irregular profile. The summit, however, was lowered 240 ft., and the maximum grade was obtained by tunneling 3 400 ft. Not until the first part of April, 1900, did the company finally decide to construct, and appointed D. C. Dunlap, M. Am. Soc. C. E., as Resident Engineer in charge of the entire cut-off, with headquarters at Laramie. Under the supervision and direction of Mr. J. B. Berry, Chief Engineer, the location at the west end was changed and the cutting at the east end deepened so as to make the length of tunnel 1 800 ft. instead of 3 400 ft.

On April 18th the contract to build the line was awarded to Kilpatrick Bros. & Collins, with a time limit of two years, and a bonus if finished in one year (which meant eight months). This firm afterward sublet the greater part of the work, reserving for itself that portion from Station 978, the west portal of the tunnel, to Station 1 098, these 120 stations being the most difficult part of the entire cut-off.

The writer had immediate charge of what is known as the Second Division, extending from Station 898 to Station 1 109, and his remarks will be confined to operations along these four miles, it being the purpose to present as concise a description as possible of the heaviest part of this very heavy work.

The curvature on this division was comparatively light, the maximum being 3°, with a total of 142° 03′ in angles, divided between four curves; but the grading was extraordinary, the quantities being as shown in Table No. 1.

TABLE No. 1.

Class of Work.	Section No.				Summary.
	18	19	20	21	
Embankment ..... (cu. yds.)	81 545	.....	7 225	723 755	812 525
Earth excavation..... " "	3 860	27 215* / 27 300†	36 391	.....	94 766
Solid granite excavation..... " "	13 214	130 330* / 105 039†	52 569	11 105	312 257
1 800-ft. tunnel, solid granite. " "	.....	35 000	.....	.....	35 000
Totals ..... (cu. yds.)	98 619	324 884	96 185	734 860	1 254 548

* West tunnel approach. † East tunnel approach.

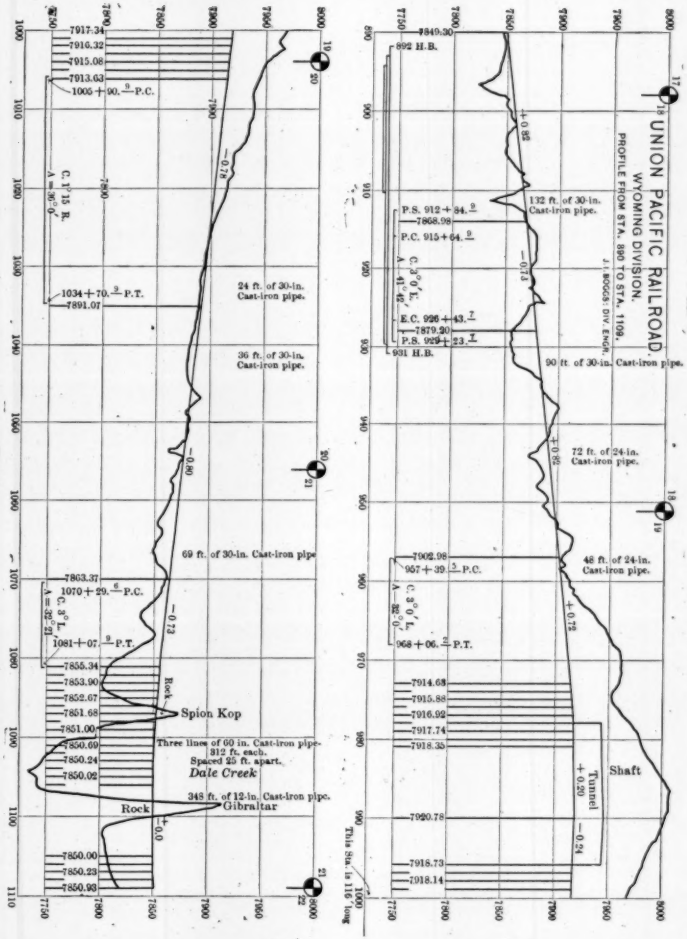

UNION PACIFIC RAILROAD
WYOMING DIVISION.
PROFILE FROM STA. 890 TO STA. 1109.
J. I. BOGGS, DIV. ENGR.
7849.30
892 H.B.
+0.95
P.S. 912+34.9
7868.98
P.C. 915+64.9
C. 8° 0' L.
A = 41° 42'
E.C. 926+43.7
7879.20
P.S. 929+23.7
931 H.B.
132 ft. of 30-in. Cast-iron pipe.
−0.74
90 ft. of 30-in. Cast-iron pipe.
+0.85
72 ft. of 24-in. Cast-iron pipe.
7902.98
957+39.5 P.C.
C. 8° 0' L.
A = 32° 0'
968+06.9 P.T.
+0.75
48 ft. of 24-in. Cast-iron pipe.
7914.63
7915.88
7916.92
7917.74
7918.33
Tunnel
+0.20
−0.24
Shaft
7920.78
7918.73
7918.14
This Sta. is 116 long
7917.34
7916.32
7915.08
7913.63
1005+90.9 P.C.
C. 7° 15 R.
A = 36° 0'
−0.78
1034+70.9 P.T.
7891.07
24 ft. of 30-in. Cast-iron pipe.
36 ft. of 30-in. Cast-iron pipe.
−0.80
69 ft. of 30-in. Cast-iron pipe
7863.37
1070+29.6 P.C.
C. 8° 0' L.
A = 32° 0'
−0.72
1081+07.9 P.T.
7855.34
7853.90
7852.67
7851.68
7851.00
Rock
Spion Kop
Three lines of 60 in. Cast-iron pipe. 312 ft. each. Spaced 25 ft. apart.
7850.69
7850.24
7850.02
Dale Creek
348 ft. of 12-in. Cast-iron pipe.
Rock
Gibraltar
+0.0
7850.00
7850.23
7850.93

Fig. 1.

Of the 723 755 cu. yds. of embankment on Section 21, 475 268 cu. yds. were in the fill at Dale Creek, between Stations 1 088 and 1 098. See Fig. 1.

The work was divided as follows:

Station 898 to 907, Mahoney Brothers; Subcontractors.
" 907 to 925, G. C. Smith; "
" 925 to 971, Mahoney Brothers; "
" 971 to 978, Beaumier Contracting Company; Subcontractors.
" 978 to 1 098, Kilpatrick Brothers & Collins; principals.
" 1 098 to 1 100, Rowan & Scott; Subcontractors.
" 1 100 to 1 109, Cushing & O'Keif; Subcontractors.

This being on the summit, there was no bridging to contend with, all water courses being taken care of by means of 24 and 30-in. cast-iron pipe, with the exception of Dale Creek, where three lines of 60-in. cast-iron pipe were laid in concrete. This pipe was in 12-ft. lengths, 2½ ins. thick, and its weight averaged 1 270 lbs. per foot. Of this 60-in. pipe 936 ft. were placed at elevation 7 749, thus impounding 15 ft. of water for a pumping station, Dale Creek being the only constantly flowing stream, of any amount, on the division. The contractors were fortunate, however, in finding innumerable minor springs of very good water all over the country.

Snow storms were of daily occurrence and the ground was covered with snow from 1 to 6 ft. deep on April 21st, when Messrs. Kilpatrick Brothers & Collins began the erection of their camps. One camp was established opposite Station 1 005 and a second on Dale Creek. On April 30th the writer arrived, and found that for two days there had been a violent snow storm. The camps, however, were in good shape and the contractors were ready and anxious to begin work.

At the outset the contractors had to do much preliminary work; the building of a spur track two miles long for the ready delivery of material from the main line of the Union Pacific Railroad; blacksmith and machine shops to be erected; electric light and compressed air plants, with their attendant wire and pipe lines, to be installed; a shaft for the tunnel to be started at once; nearly two miles of double-track 3-ft. gauge railway to be built; a pumping station to be placed on Dale Creek, with a pipe line extending to Station 960; besides many other important items, all demanding immediate and close attention, with not a day's time to lose, as the seasons in this altitude and lati-

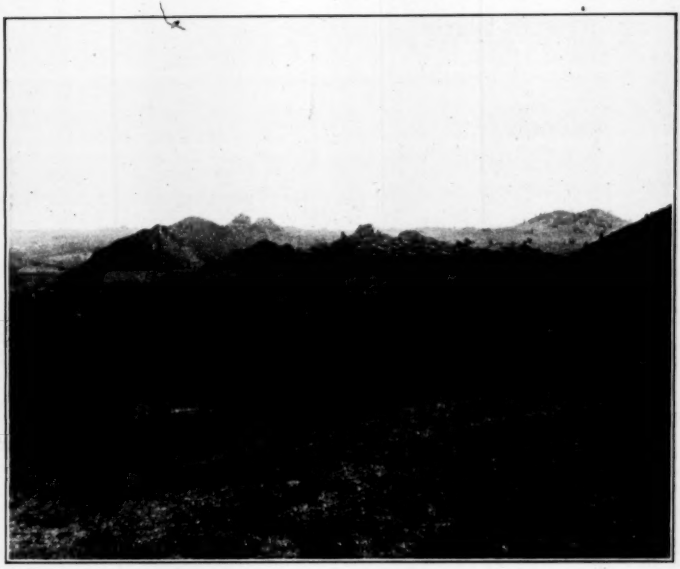


FIG. 1.—GENERAL VIEW OF HEAVIEST PART OF WORK; U. P. RY., IN WYOMING.

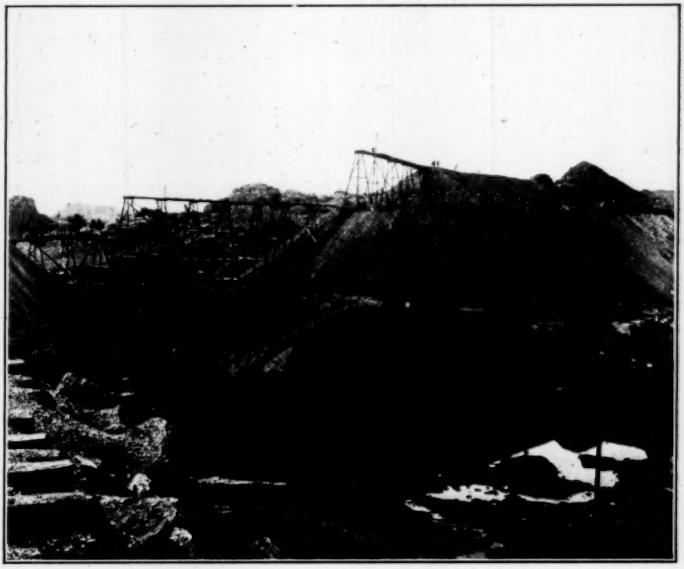


FIG. 2.—DALE CREEK FILL, U. P. RY., IN WYOMING.

tude are very short, the last snow storm having occurred on June 9th and the first of the next season on September 26th.

The first efforts at actual construction were directed toward getting a number of teams on the grade between Stations 1 030 and 1 069, and the writer's first duty was to cross-section between these points. On May 1st and 2d, in the midst of heavy squalls of snow, the engineer corps cross-sectioned the rock hill dubbed "Spionkop," between Stations 1 086 and 1 088, fifty rock men being started there at once.

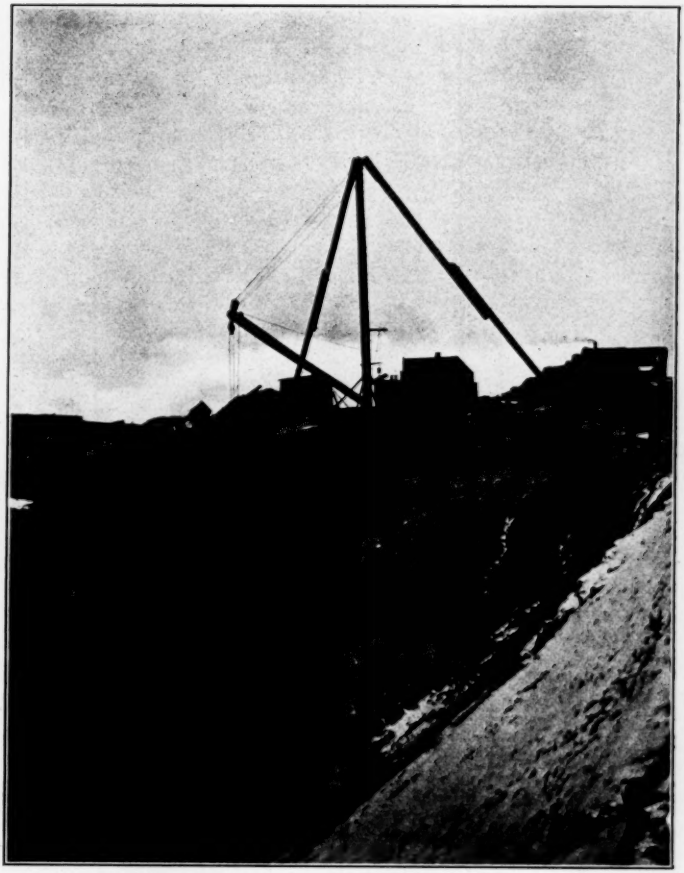
In the morning of May 4th the east tunnel approach was cross-sectioned from Stations 996 to 1 023, and at 1 P. M. of the same day the excavation was commenced.

On the following day cross-sections were run from Stations 971 to 978, being part of the west tunnel approach, and work was started immediately after the stakes were driven, in fact, before all were in. At 5 P. M. of this day the first steam shovel arrived. On May 7th the foundations for the electric light and compressor plants were begun, and five laborers, with a foreman, began sinking a shaft for the tunnel at Station 984. On the same day Mahoney Brothers commenced the erection of their camps opposite Station 955. May 15th may be set as the date of the beginning of actual operations, for, although a number of teams had been working on the grade previous to this date, it was not until this day that the steam shovels were started, one being placed in a borrow pit opposite Station 1 060 and about 350 ft. to the south, a second beginning in the east tunnel approach at Station 1 004. At this time the double-track railway was completed as far as Station 1 070. Fig. 1, Plate I, is a general view of that portion of the work lying between Stations 1 070 and 1 098, "Spionkop" lying a little to the left of the center, with the "Rock of Gibraltar" beyond and to the left, the grade line being shown on the former by the cutting through its peak and on the latter by the working bench, while between the two lies Dale Creek and the famous fill of 475 000 cu. yds.

From this time on, machinery was rapidly put in place, until finally there were in operation eight steam shovels, and fifteen locomotives with trains of from ten to eighteen 3-yd. dump cars each. The machine drills were run by compressed air, and work was carried on both night and day, the works being lighted by electricity.

From May 15th to June 1st steam shovels and teams were engaged in building the embankment, from Stations 1 070 to 1 085, up as high

as the lower level shown in Fig. 1, Plate I, this level striking "Spionkop," near the 7 825-ft. contour, or 27 ft. below grade. On June 4th a line following this contour as closely as possible was located around the hill for the 3-ft. gauge railway, the curves running from 40 to 70 degrees. This is partly shown in Figs. 1 and 2, Plate I. At Stations 2 + 71 and 4 + 73 on this narrow gauge line, split switch points were inserted, and about June 8th the active construction of the immense fill may be said to have been begun, by side-dumping and constantly throwing the track ahead. This side-dumping was continued until about July 19th, when, owing to the failure of the foundries to supply the 60-in. pipe promptly, the work on the west side of the creek had to be temporarily stopped. The contractors then commenced filling to the grade line west of "Spionkop," shown as the upper level in Fig. 1, Plate I. Prior to this, however, a line had been located and constructed on the 7 790 ft. contour, from a borrow pit on the east side of Dale Creek, around the base of "Gibraltar" to the south line of slope stakes. Here, one shovel and two trains were constantly pouring in material, in the quadrant of a circle whose radius was the distance between the opposite slope stakes; the track being occasionally lengthened by means of temporary trestles, one of which may be seen on the left in Fig. 2, Plate I. The location of a part of this line may also be seen in Fig. 1, Plate I. It was not until July 30th that the 60-in. pipes were in place, and work was resumed on the west side. By this time the bank west of Station 1 086 had been filled to grade, as shown in Fig. 1, Plate I, and it became necessary to remove the track from the south side of "Spionkop," disconnecting it at Station 2 + 71 and resorting to filling, in the quadrant of a circle, as was done on the east side. This continued until Station 1 093 was reached, when trestling was used, as shown in Fig. 2, Plate I. The height of the trestling, however, was reduced 20 ft. by having a number of teams build earth bases before its construction was commenced. On August 1st the narrow-gauge track was laid through the rock cut on the top of "Spionkop," and the first bent of temporary trestle for the upper and final lift was erected. Fig. 2, Plate I, shows the condition of the work on September 1st; "Spionkop" being on the extreme right, the first trestle to the left being to grade, the second 30 ft. below, and the one on the extreme left, coming from the shovel on the east side, 60 ft. below grade. December 1st saw this gigantic



PORTAL OF SHERMAN TUNNEL, U. P. RY., WYOMING.

bank of 475 000 cu. yds. completed. This fill is to take the place of a bridge on the main line 127 ft. high. The writer did not agree entirely with the foregoing method of construction, as it was productive of much expense, annoyance and delay. He is of the opinion that had a line been built around "Spionkop" on the 7 790-ft. contour with trestling from the first, it would have been found much more expeditious and less expensive; however, he was overruled in the matter at the time, but since then has had the poor satisfaction of having the contractors tell him that they regretted not having adopted his plan.

As stated, on May 7th, work was begun on a shaft for the tunnel at Station 984, and was continuously prosecuted, night and day, through the solid granite, until on June 13th a depth of 65 ft. was reached, 5 ft. below the springing line of the arch. Here the first difficulties and anxieties of the engineers in charge began, for it now became necessary to set grades and centers for the two headings; one working east, the other west. The writer had no suitable instruments, the best being an ordinary Gurley's plain transit, and a 20-in. level. He also had two 50-ft. metallic tape lines, and two 1-lb. plumb-bobs of cast iron, having points $\frac{1}{8}$-in. in diameter. Placing two 3 x 12-in. planks securely over the shaft, two nails were set on center and partially driven. A wire, fastened at each end to the nails, was then stretched across, and the two plumb-bobs were suspended therefrom. Two holes, one on each side, were then drilled 1 ft. above the crown, two wooden plugs were inserted, the plumbs were again dropped and two nails set. For elevation, two metallic tape lines were tied together and the elevation was measured in from a bench set at the edge of the shaft and checked by means of a wire measured to exact length over the level rod. Hoisting engines, gallows frame, and blacksmith shops were soon erected, and in a short time the center line was completely obstructed, so that when the headings were driven about 75 ft. on each side, and centers had to be reset, the first having been blown out, it became necessary to use an offset line. This, the writer disliked doing, owing to the poor character of the instruments at hand; however, it was imperative and he had no choice in the matter. At Station 992, east of the shaft, a right angle was turned and an offset line 60 ft. to the south established, and carefully checked in on a point at Station 982, to see that it was parallel. A point was then established opposite the shaft and a line run

across it at right angles to the direction of the headings. Two nails were again set, a wire stretched between and the two plumb-bobs dropped 9 ft. apart. A distance of 55½ ft. was carefully measured to the first and a point set beneath it in the floor of the shaft; the second was allowed to remain suspended as a foresight. Right angles were then turned and an offset line 4½ ft. south of the center was run along the floor of the headings in both directions. For the center, points in the roof were then set from this line, and plumbs suspended from them. For a check, as 3 ft. to the north would clear the obstructions east of the shaft, an offset line at that distance was run across it, and the two plumbs suspended with 6 ft. between them, a point being taken below one; the other remaining suspended, as before, for a sight. A 3-ft. offset line was run along the floor of both headings, and the points in the roof, previously set, were checked from this. These points checked fairly well, and were thereafter used to prolong the line when necessary.

The operations noted, while simple enough above ground, are very different beneath it, in a small comparatively dark hole, working by candle light, with the clatter of machine drills on all sides, gangs of "muckers" waiting, etc., especially when only very common field instruments are available. Unless the engineer in charge has a cool, clear-headed instrumentman he is likely to experience considerable trouble where the headings meet.

On one occasion, the instrumentman declared that nothing more could be done, as the cross-hairs had slackened so badly, from the surrounding moisture, that he could not set a point within ¼ in. Something more had to be done, as it was imperatively necessary to set a center ahead; so, in desperation, the writer picked up a couple of candles and placed the flame under the diaphragm of the cross-wires. This dried and straightened them up, and, with occasional applications of the candle flame, the work proceeded.

On July 6th the heading east from the west portal at Station 978, was started, and on August 23d, at Station 981 + 37, it met the heading being driven west from the shaft. The center line was then carefully checked up and was found to be exactly correct at Station 984, at the bottom of the shaft; while, where the two headings met, it was found that the line from the shaft was precisely 1 in. too far north, showing that a slight angle had been obtained in dropping the plumbs.

FIG. 1.—EAST APPROACH TO SHERMAN TUNNEL, LOOKING EAST.

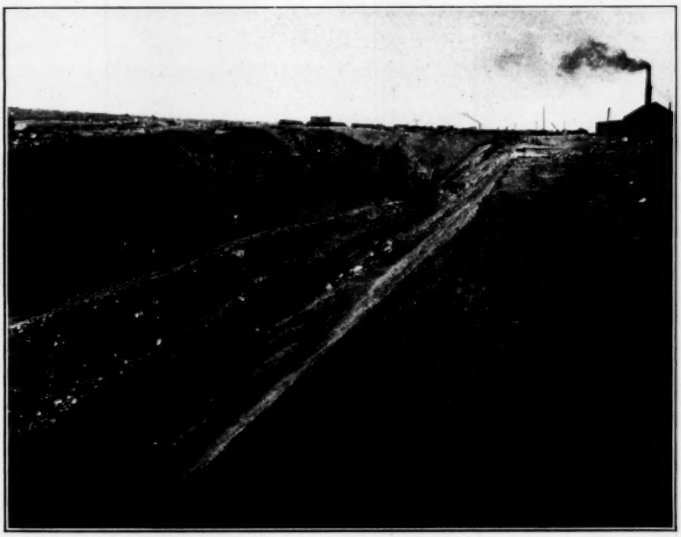


FIG. 2.—EAST APPROACH TO SHERMAN TUNNEL, LOOKING WEST.

On July 26th the heading west from the east portal was started and continuously driven night and day until on December 8th, at Station 988 + 37, it met the heading driven east from the shaft. Work on the latter heading had been stopped on September 26th, after being driven 437 ft. At that time the force was put on the bench. The entire time occupied in driving the headings is shown in Table No. 2.

TABLE No. 2.

	Days.	Number of feet driven.	Average rate, in feet, per day.
West Portal	49	337	6.88
West Shaft	72	263	3.65
East Shaft	106	437	4.12
East Portal	139	763	5.49
Totals and average	366	1 800	4.92

Usually, six drills on three columns were worked in the headings, the cut being drilled 8 ft. and the round 10 ft. The most difficult and treacherous rock encountered was between the shaft and the west portal, and all this part will have to be timbered; while from the shaft to the east portal the material was very firm. The bench was taken out in two lifts, and the progress has been as shown in Table No. 3.

TABLE No. 3.

	Days.	Feet.	Average rate, in feet, per day.
West Portal	92	373	4.06
West Shaft	91	227	2.49
East Shaft	97	453	4.67
East Portal	51	180	3.53
Totals and average	331	1 233	3.73

Much time was lost between the completion of the heading and the starting of the bench at the west portal, as the entire heading had to be slowly and carefully timbered. As shown by the foregoing figures, 567 ft. of bench yet remain to be taken out, between the shaft and the east portal; this, at the present rate of progress, will require about 70 days' more time. It is hoped, however, that this estimate will be materially reduced.

The forces employed in driving this tunnel, so far, reduced to a unit of one day, together with the wages paid, are shown in Table No. 4.

TABLE No. 4.

	Force, reduced to a unit of one day.	Wages paid.
Drillers	7 230	$3.25 per day
Drill helpers	8 710	2.25 "
Muckers	19 980	2.25 "
Foremen	1 236	125.00 per month
Total of all classes	37 156	

No water, of any amount, has been encountered. Plate II illustrates the method used in hoisting muck at the west end; it being then run out to the waste bank by cars. The frame-work on the right is the dumping frame for the skips. At the shaft the muck was hoisted in cages, in 1-yd. steel cars, which were run out on the waste bank and dumped. At the east portal, track was laid in as the work progressed, and the muck was run out through the east approach, and wasted.

Fig. 1, Plate III, is a view of the east approach, showing the condition of the work on July 23d. Fig. 2, Plate III, is another view of the same on August 25th. The tunnel heading can be seen near the center; on the right are the machine and blacksmith shops, the electric light and compressed air plant, known as "The Power House," while over the hill, and above the excavation, is the top of the smoke-stack of the hoisting engine and gallows frame at the shaft.

On May 22d Roman & Scott, with a force of eleven men, about all that could be worked to advantage, began excavating the deep rock cut at the hill dubbed "Gibraltar." This was a peak of solid granite, 70 ft. above the grade line at the highest point. Operations commenced by running a 3 × 4 ft. tunnel along the center line and at grade for 50 ft. An ell, 20 ft. long, was then drifted to the north, and charged with 8 750 lbs. of powder. Fig. 1, Plate IV is a view of the original hill with the tunnel entrance a little to the right of the center. This excavation was completed on November 1st.

On May 24th Cushing & O'Keif began on their 110 000-yd. embankment, 900 ft. long, and finished it on October 20th.

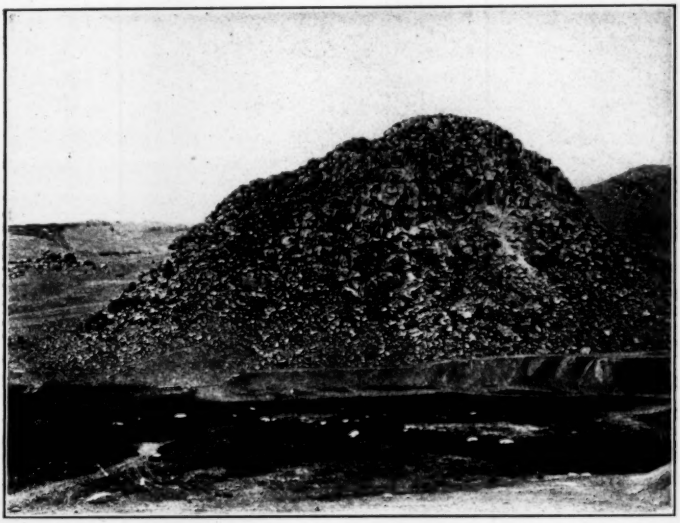


FIG. 1.—"GIBRALTAR," U. P. RY., IN WYOMING.

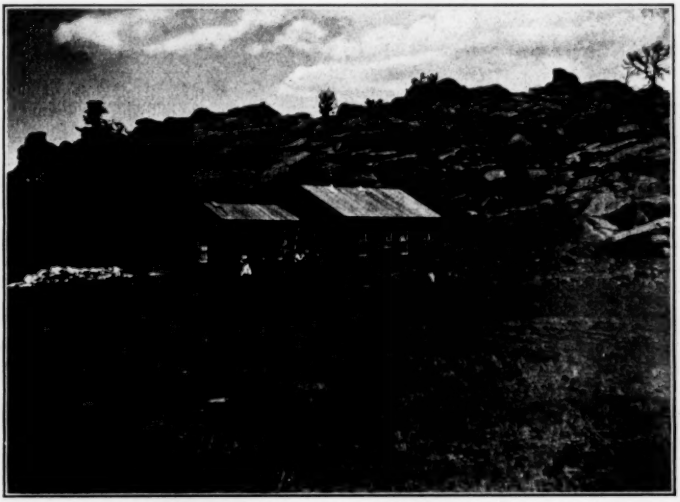


FIG. 2.—ENGINEER'S HOUSE, U. P. RY., IN WYOMING.

On June 16th the Beaumier Contracting Co. started on their 60 000-yd. excavation, 86% of which was solid rock, and finished it on December 31st.

On May 10th Mahoney Bros. started on their sub-contract, and will probably finish in February, 1901. Nearly all their excavations have been through the very hardest granite, and considerable difficulty has been encountered in moving it, on account of the immense amount of drilling necessary before the steam shovels could properly handle it. All the shallow excavations between Stations 907 and 925 consist of hard granite, and the roadbed being 47 ft. wide, on account of side tracks, made it very difficult; however, it was well handled. On July 31st Mr. Smith began by drilling holes 4½ and 5 ft. apart all over the surface, and on October 15th had finished the entire eighteen stations.

Out of 1 250 000 cu. yds. to be moved on the 4 miles, only 12 000 cu. yds., exclusive of the tunnel, remained on December 31st, 1900.

The specifications admitted of but two classifications, "Earth" and "Rock." All that could not be ploughed with a No. 1 breaking plow and six horses had to be classed as solid rock. To draw the line between the two was sometimes very difficult.

The writer does not claim anything extraordinary about this work, except the wonderfully short time in which an immense amount of railway grading can be accomplished with modern appliances and methods under difficult and discouraging conditions. One of the greatest annoyances to which the contractor is subjected in this section is the worthless, thriftless class of men he has to depend upon for labor.

Sections 18, 20 and 21 were entirely finished on December 31st, 1900, and final estimates were rendered. On Section 19 Mahoney Bros. still have about 12 000 cu. yds. to move and some sloping to do. Beaumier has to do some slope trimming, and not quite one-third of the bench in the tunnel has yet to be moved, while in the east approach about one-half the slopes have to be trimmed up and the cut ditched —the latter being difficult, owing to the frozen condition of the ground.

A profile of this division is shown in Fig. 1. From the start to the present time the forces, reduced to a unit of one day, have been as shown in Table No. 5.

TABLE No. 5.

Contractor.	G. C. Smith.	Mahoney Bros.	Beaumier Contracting Co.	Kilpatrick Bros. & Collins, and Wood Bros.	Roman & Scott.	Cushing & O'Keif.
Men.						
Foremen	230	1 046	704	6 854	203	240
Laborers	2 115	13 710	10 354	50 922	1 117	1 957
Drillers	385	1 850	527	7 625	307	.....
Machinemen	116	1 712	1 364	19 243	.....	.....
Trainmen	.....	1 488	.....	7 263	.....	.....
Totals	2 846	19 806	12 949	91 907	1 627	2 197
Teams.						
Scrapers	920	1 016	452	4 854	9	2 952
Carts	134	.....	.....	.....	216	.....
Plows	144	235	77	610	.....	295
Wagons	.....	305	.....	520	.....	.....
Cars	.....	24	839	1 521	.....	107
Totals	1 198	1 580	1 368	7 505	225	3 354

G. C. Smith's Plant.

1 12-H.-P. portable boiler.

2 machine drills and a number of hand drills.

Mahoney Brothers' Plant.

1 55-ton Vulcan steam shovel; 60 H.-P., geared up to 100 H.-P.; capacity of dipper, 2 cu. yds.

1 80-ton Vulcan steam shovel; engines, 13 x 15-in.; single engine, 46 H.-P.; double engine, 96 H.-P.; geared up on gear and pinion four times and on hoisting chain twice; capacity of dipper, 3½ cu. yds.; boiler, 100 H.-P.

1 Barnhart, model "A," shovel. (The work done by this shovel was insignificant.)

1 10-ton locomotive; cylinders, 8 x 14 ins.

1 12-ton " " 9 x 14 ins.

50 3-yd. Western dump cars.

2 12-H.-P. stationary boilers.

1 10-H.-P. portable boiler.

6 machine drills.

1 complete derrick.

There were 267 steam-shovel days, and 384 locomotive days, with

3 421 cars in service. No record of the average car output was kept. The average length of haul was about 2 700 ft.

Beaumier Contracting Co.'s Plant.

1 45-ton Barnhart, model "A," steam shovel, of 50 H.-P., geared up to 75 H.-P.; capacity of dipper, 1½ cu. yds.
50 1-yd. dump cars, hauled by teams.
2 machine drills.
1 complete derrick, with 12-H.-P. stationary engine.
Shovels, picks, etc.

There were 131 steam-shovel days, with 3 206 cars in service. The average haul was 1 400 ft.

Plant of Cushing & O'Keif.

1 377 1-yd. cars.

Plant of Kilpatrick Brothers & Collins, and of Wood Brothers.

Machinery in Shops.

3 100-H.P. boilers.
1 air compressor, 250 H.-P.
1 " " 200 H.-P.
2 receivers (air usually carried at 6 atmospheres).
3 blacksmiths' forges.
1 steam hammer, 100 lbs., 2-ft. drop.
1 Am. Tool Co. lathe, 36-in. swing, 8-ft. centers.
1 " " " 24-in. " 16-ft. "
1 planer, 14-ft. bed, 3 ft. under head.
2 drill presses, 5½ ft. and 3 ft.
1 bolt cutter, ½ in. to 1½ in.
1 Rice's automatic engine, 9 x 15 ins.
1 30-H.-P. high-speed engine.
1 30-H.-P. Atlas engine.
1 72-H.-P. dynamo, 250 volts.
1 50-H.-P. dynamo, 125 volts.
1 40-H.-P. boiler.

Machinery at Shaft.

1 Lidgerwood double hoisting engine, cylinders 10 x 16 ins., with double drum.
1 75-H.-P. boiler.
2 steel shaft cages, 7 500 lbs. each.
8 1-yd. steel cars.
1 blacksmith's forge.

Machinery at West Portal.

1 pair of hoisting engines, 7 x 9 ins.
1 40-H.-P. boiler.
1 blacksmith's forge.
1 complete derrick.

Steam Shovels.

1 90-ton Barnhart, Model "K"; boiler, 100 H.-P.; engines, 13 x 15 ins.; single 46 H.-P.; double, 92 H.-P.; geared up on gear and pinion four times and on hoisting chain twice; capacity of dipper, 3½ cu. yds.
1 60-ton Barnhart, Model "G"; boiler, 75 H.-P.; engines, 12 x 14 ins.; single, 30 H.-P.; double, 60 H.-P.; geared up on gear and pinion four times and on hoisting chain twice; capacity of dipper, 2½ cu. yds.
1 70-ton Bucyrus, No. 350; boiler, 75 H.-P.; engines, 10 x 16 ins. single, 35 H.-P.; double, 70 H.-P.; geared up on gear and pinion five times and on hoisting chain twice; capacity of dipper, 2½ cu. yds.
1 50-ton Bucyrus, No. 157; boiler, 70 H.-P.; engines, 10 x 14 ins.; single, 30 H.-P.; double, 60 H.-P.; geared up on gear and pinion five times and on hoisting chain twice; capacity of dipper, 2½ cu. yds.

On the Works, Generally.

7 12-ton Porter engines; cylinders, 9 x 14 ins.
1 10- " " " 8 x 11 "
1 10- " Porter & Bell engine; cylinders, 10 x 12 ins.
2 15- " Baldwin engines; cylinders, 10 x 16ins., 6-wheel connected.
1 15- " " " 10 x 15 " " "

4 Fairbank's M. & Co. pumps, 6 x 4 x 6 ins.
250 3-yd Western dump cars.
50 1-yd. dump cars.
25 machine drills.
3 water tanks; 8 000 galls. each.
5 wagon tanks.
2½ miles of 3-in. iron pipe.
3 600 ft. of 4-in. iron pipe.
2 000 " 2½-in. "
300 " 1½-in. "
1 000 " 1-in. "

Also a number of shovels, picks, lanterns, cables, etc., etc.

The actual shovel output is shown in Table No. 6, together with the average haul, etc.

TABLE No. 6.

Shovel.	Cars.	Average haul.
Bucyrus, No. 157	14 094	2 000 ft.
Bucyrus, No. 350	61 436	8 000 "
Barnhart, Model "K"	65 791	9 500 "
Barnhart, Model "G"	58 221	1 600 "
Total	199 542	.....

There were 472 steam-shovel days and 1 479 locomotive days, with 17 898 cars in service.

The quantity of powder used by Kilpatrick Brothers & Collins on their portion of the work, between Stations 978 and 1 098, was as follows:

Gelatine	106 700 lbs.
Dynamite (50%)	49 350 "
Judson powder	10 600 "
Black powder	139 250 "
Total	305 900 "

For skilled labor the rate was generally $3 per day; for unskilled labor $2 per day, and for foremen $100 per month.

There have been few accidents, considering the magnitude of the

work. Three men have lost their lives, and less than a dozen have been injured, only two of these seriously.

Too much praise cannot be given to the contractors, Messrs. Kilpatrick Brothers & Collins, and Wood Brothers, for the vigorous manner in which they carried out the work.

Fig. 2, Plate IV, exhibits the style of houses built by the Union Pacific Railroad Company for the use of the Division Engineers and their parties. On the right of the main building is a large room, accommodating eight men with comfort; in the center is the living room and kitchen; on the left is the engineer's office and bedroom. The ell consists of two rooms for the accommodation of the engineer's family. The house was double-walled, with tar paper between, and closely ceiled.

No trouble or expense was spared by Mr. Dunlap in making the men under him comfortable; the house was well and completely furnished by the company, and there was a plentiful fuel supply, which is greatly needed in that severe climate. The house was lighted by electricity from the "Power House."

The writer desires to return thanks to his instrumentman, Mr. A. J. Wharf, for his careful, patient and painstaking work, which made it possible to accomplish such good results.

It may seem that this paper is premature, because the work is not yet finished. The writer regrets this, and offers in excuse the fact that the present seemed to be the only time which he could devote to its preparation.

DISCUSSION.

EMILE LOW, M. Am. Soc. C. E. (by letter).—The writer desires to draw attention to some heavy railway construction in Virginia, on the line of the Clinch Valley Division of the Norfolk and Western Railroad (now Railway). This division extends from Graham, on the Norfolk-Columbus Division, to Norton, the junction with the Louisville and Nashville Railroad, a distance of 100 miles. The line traverses the mountain region of Southwestern Virginia, forming the western slope of the Allegheny Mountains. Some of the mountain peaks in this section reach an altitude of nearly 5 000 ft. above sea level. The line generally follows Wright's Creek (a tributary of the New River), Clinch River, Thompson's Creek and Guest's River, the two latter being tributaries of Clinch River. Very heavy work occurs at the divides where the line passes from one valley to another; especially is this the case between St. Paul and Colburn. The cost of the heaviest sections, usually one mile in length, in round numbers, was as follows: Section 57, $100 000; Section 81, $121 000; Section 85, $166 000; Section 86, $114 000; and Section 89, $153 000. Six tunnels, ranging in length from 327 to 1 925 ft., occur on the sections mentioned, hence the large cost. Mr. Low.

On the hundred miles there were 25 sections the cost of which ranged from $40 000 to $166 000, with a total cost of $1 718 000, or an average of nearly $70 000 per mile, the cost of the track not being included.

It is the writer's intention, however, to mention more specifically the construction of Sections 31 to 35, inclusive, between Pounding Mill and Cedar Bluff, where the construction consisted almost wholly of open excavation and bridging. On these sections there are only three short tunnels, 155, 437 and 540.5 ft. long, respectively. The railway between the points mentioned follows the valley of the Clinch River, a tortuous mountain stream, which cuts its way through a range of limestone hills, known locally as Kent's Ridge. The river is crossed ten times. Near Pounding Mill five crossings occur within half a mile; the other five crossings occurring within the last 6 000 ft., near Cedar Bluff.

The course of the Clinch River east of Pounding Mill is comparatively straight, but the south bank is an almost perpendicular cliff for the distance mentioned. The course of the river just west of Pounding Mill is very sinuous, the distance between Bridges 1338 and 1342 (known as Crossings 9 and 13, of Clinch River), measured along the center line of the railway is 2 700 ft., while the length of the stream between the same points is about 7 600 ft., or nearly three times as long.

As previously stated, the river is crossed five times in about half a

Mr. Low. mile. The crossings range from 33 to 44 ft. in height, between base of rail and water surface. The bridges are Howe trusses, resting upon masonry substructures. For the higher bridges, trestles are used for the approaches on either side of the main span.

There being practically no embankment on this 2 700-ft. section, nearly all the material from the excavations had to be wasted. The narrow ridge between Bridges 1340 and 1341 is pierced by a short tunnel known as Tunnel No. 1.

Some of the bridges near Cedar Bluff are located on curves, the others on tangents, and T-walls were here substituted for the short spans at the ends.

The strata in the section mentioned dip to the south, the dip ranging from 30 to 45 degrees. This fact largely increased the quantities of excavation, as the cuts, upon being excavated, assumed slopes on one side corresponding to the dip of the rock, the excess material beyond the prescribed slope lines being literally solid rock slides—an uncommon kind of slide.

In one case a slide of this character occurred after the track was laid. This was in the cut between Bridges 1338 and 1339. One night, during a heavy rain, the upper side of the cut slid in, covering the track to a depth of over 20 ft., for a length of about 400 ft., about 90% of the slide being composed of solid rock.

The heaviest work occurred on Section 31, where the south bank formed an almost perpendicular wall over 110 ft. in height, from the surface of the river up.

Section 33, comprising what was called the Horse Shoe Bend, was the lightest.

Tunnels Nos. 2 and 3 were located on 8° curves. As the tunnels were worked from both ends it was very essential that the alignment should be correct. Previous to construction the center line was carefully re-run, all measurements being made with steel tapes. At convenient places outside of the tunnel portals, points on the center line were established, and lines tangent to the curve run out to intersections. The tangents were carefully measured, as also the long chords and versed sines. After all measurements were found to agree with the theoretical calculations, the line was carefully referenced and construction started. It is quite needless to say that the work from each end met, absolutely.

The price paid for tunnel excavation, proper, was for the material only between the neat lines of the prescribed cross-section. On account of the inclination of the strata passed through, there were more or less falls, or breakage, as it was termed. The contractors were paid for this excess material, beyond the prescribed section, at a reduced price. The percentages of this breakage, as compared with the neat or prescribed sections, for the different tunnels were as follows:

No. 1, 19%; No. 2, 16%; No. 3, 24 per cent. These percentages of falls may seem large, but were wholly attributable to the inclination of the strata, as before mentioned. Mr. Low.

Hand-drilling was largely used in excavating these tunnels, although steam drills were used occasionally.

The tunnels required no timbering, except that a short length of timber lining was put in at the west portal of Tunnel No. 2, where the material was subject to disintegration.

There was no contract price for foundation, coffer-dams and other items connected with bridge foundations. This and other work was termed "extra work" and was paid for by force account, the contractor being paid the actual cost of labor and materials, plus 15% for use of tools and superintendence.

Each day the contractor furnished the resident engineer with an itemized statement of all the extra work done on the previous day. This statement was made out on blank forms supplied by the railway company, and showed in detail the kind of extra work performed, the locality, the number of workmen of all classes engaged, quantities of material used, rates of pay and of material, and the money amount for the day. A separate report was made for each piece of work. These daily reports were duly checked by the resident engineer, and all discrepancies, if any, harmonized and corrected at once.

A weekly summary of all extra work performed was sent to the division engineer for inspection and approval, and also for information, so as to enable him to modify or change any particular piece of work, if the expense seemed unreasonable.

Monthly bills were prepared from the daily reports, each piece of work being duly itemized and charged to the section to which it belonged; this method being preferred to the somewhat usual practice of converting the amounts into quantities of excavation and thus being lost to view in the general result. Table No. 7 gives in detail the quantities and cost of the work.

TABLE No. 7.

Section No.	Open Excavation.				Tunnel Excavation.		Masonry, all classes.	Clearing and grubbing.	Cost, including bridges.
	Earth.	Loose rock.	Solid rock.	Total.	Regular.	Breakage.			
	Cu.yds.	Cu.yds.	Cu. yds.	Cu. yds.	Cu.yds.	Cu.yds.	Cu.yds.	Acres.	
31.....	6 347	4 922	85 623	96 892	.....	.....	928	7.09	$76 000
32.....	10 599	8 477	63 022	82 098	1 514	288	1 019	9.61	82 000
33.....	1 100	7 271	27 029	35 400	.....	.....	146	12.19	24 000
34.....	4 924	4 619	31 171	40 714	5 252	821	1 520	5.69	66 000
35.....	6 651	2 560	26 528	35 739	6 126	1 467	1 742	4.47	70 000
Totals..	29 621	27 849	233 373	290 843	12 892	2 576	5 355	39.05	$318 000

Mr. Low. The cost of Section No. 36 was $54 000.

The cost of track is not included in Table No. 7.

It will be noticed that a large percentage of the material excavated was solid rock, the earth and loose rock being merely thin coverings overlying the solid rock.

The various percentages of the excavation were: Earth, 10.2; loose rock, 9.6; and solid rock 80.2.

The following are the contract prices paid for the various items of work:

Earth excavation	$0.18	per cubic yard.
Loose rock excavation	0.35	" "
Solid " "	0.75	" "
Tunnel excavation	3.25	" "
Tunnel breakage	1.50	" "
Rock-face ashlar masonry	8.00	" "
Broken range "	6.00	" "
Box culvert "	3.00	" "
Dry paving "	1.50	" "
Rip-rap "	1.00	" "
Clearing and grubbing	30.00	per acre.

These prices were fixed by the chief engineer, and were the prevailing prices for the whole line.

The cost of the explosives for the open excavation was approximately 5, 10 and 15 cents for earth, loose rock and solid rock, respectively.

The wages paid were $1.20 per day for laborers, who were nearly all negroes, and $3 per day for foremen.

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No. 899.

THE KINZUA VIADUCT OF THE ERIE RAILROAD COMPANY.

By C. R. Grimm, M. Am. Soc. C. E.

Presented January 2d, 1901.

WITH DISCUSSION.

The Kinzua Viaduct is on the Bradford Division of the Erie Railroad, and is situated in McKean County, Pa.

The old viaduct was in service for about nineteen years, and has now been replaced by a new steel structure, built under the charge of C. W. Buchholz, M. Am. Soc. C. E., Chief Engineer of the Erie Railroad Company, Mr. M. Strong being the Bridge Engineer. The contractors for the steel work were the Elmira Bridge Company, Limited, Elmira, N. Y.; and the contractors for the erection were Grattan and Jennings, Buffalo, N. Y. The drawings were prepared under the care of E. E. Buchanan, M. Am. Soc. C. E., Secretary of the Elmira Bridge Company. The writer, in his capacity as Engineer of that company, designed the viaduct and also suggested the particular nature of its towers, which, in their construction, form a radical departure from previous practice in viaduct designing. The character of the tower bents is not new, as such forms have often

been used by structural as well as mechanical engineers; but their application to a railway structure and on a large scale is certainly a novelty.

The design of the viaduct is quite simple, so that it will suffice to give a brief description, but before proceeding to do so, it may be well to state the reasons which led the writer to make his suggestions.

The original problem consisted of the design of towers with stiff members throughout, together with secondary struts and diagonals for the taller towers, lying in a longitudinal vertical plane, coincident with the axis of the structure, and forming a support for the very long members of the wind bracing. A design for the tallest tower, in conjunction with an estimate of its weight, was prepared by the writer, but the weight proved to be excessively great, even after many changes in the design had been introduced. The next step was the substitution of lateral rods for all the longitudinal and transverse diagonals, but this design, too, although calling, of course, for less weight, was soon abandoned. In the early stages of the design the writer entertained no doubt that a tower of nearly 300 ft. in height, and based on the lines described, would never give the desired results, but that, on the contrary, they could be effected by a radical change.

Such tall towers have far too many members to conform with all those advantages which are sought to be realized in a structure. The secondary members, running to a very great weight, and solely designed as a crutch for the protection of the wind bracing, did not appear to the writer as simple means to accomplish a certain purpose. On the contrary, he thought that this purpose could be attained more simply and more directly by building the wind bracing strong enough to take care of itself without the support of a secondary bracing, and likewise, he thought that the tower legs, being the real supporters of the railway, should stand out boldly, instead of looking rather slender and more or less overshadowed by a great mass of bracings. But columns of large size meant a considerable increase in the height of the stories, fewer members to build, a quicker erection and a great reduction in field riveting.

For these reasons the writer suggested two different forms of tower bents, one, as actually built, without transverse diagonals, and the

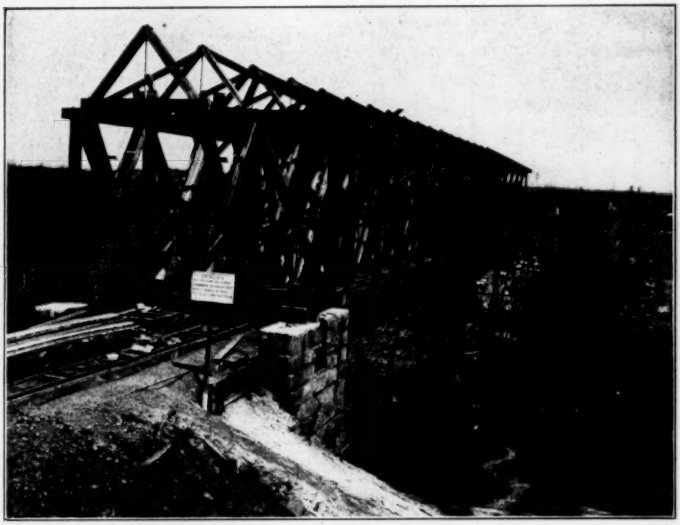


FIG. 1.—THE OLD KINZUA VIADUCT AND TRAVELER.

FIG. 2.—NEW KINZUA VIADUCT IN PROCESS OF ERECTION.

other as shown in Fig. 1, in both of which a secondary bracing is dispensed with.

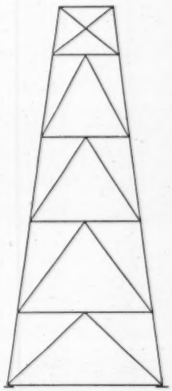


FIG. 1.

Estimates of the weight of each frame were made, and, although the rejected form of bent has merits well worth full consideration, it may be dismissed at once, as its discussion is outside the province of this paper. The following is a brief description of the work:

The great reduction of the bending moments, due to the wind forces, and a consequent reduction in the weight of the towers, is due to the fact that each bent of the accepted form is practically a triangle, with the principal wind force acting at the top, the sides being rigidly united at the apex by a header, 8 ft. 10 ins. deep, made up of two solid plates as webs, the chords of which are built of angles. These facts, taken in connection with the general appearance of the frame, have favored the adoption of this form of bent.

The viaduct, resting on the old piers, is 2 053 ft. long, and carries the railway on twenty towers varying in height from 30 ft. to about 285 ft., from the top of the masonry to the base of the rail, and having a batter of 1 to 6. Fig. 2 is an elevation of the highest bent. As mentioned before, no secondary bracing has been used, but the tallest towers are provided with horizontal adjustable bracings to line up the tower legs during erection. The dimensions, sizes and details of all the towers are the same from their tops downward, or in other words, the dimensions, sizes, etc., for any tower are obtained from the tallest tower by shortening the same from the bottom upward.

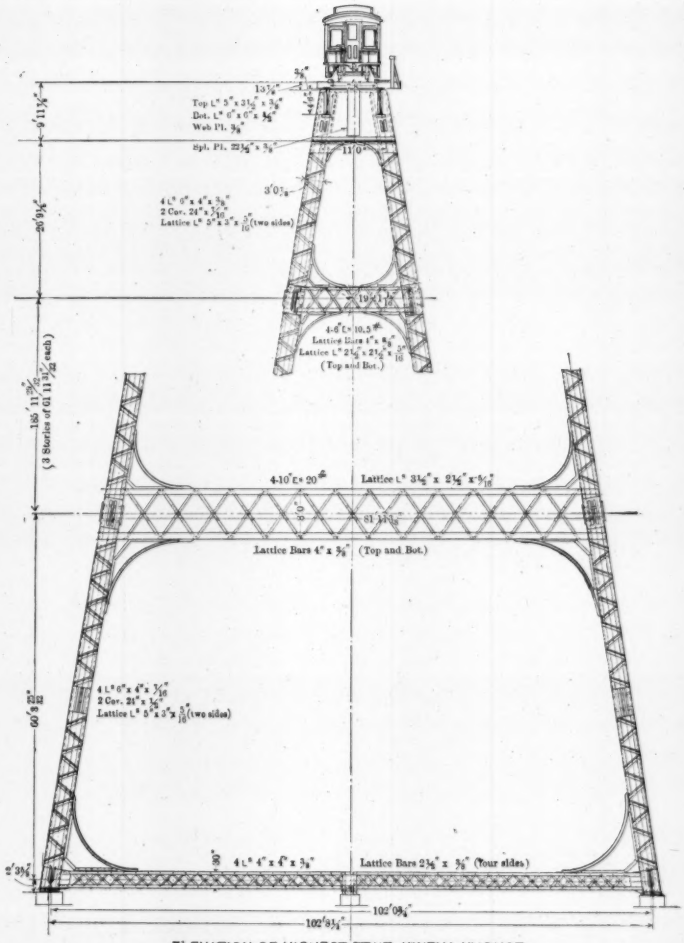
The height of the stories, taken transversely, is 62 ft., which means an increase in height of about 100% over that which is common in viaducts, while the height on the longitudinal faces is 31 ft. The columns have no bents at the top, and are composed of two plates, 2 ft. wide, and four 6 x 4-in. angles. They measure 3 ft. 1 in. transversely, and are laced by 5 x 3-in. angle bars. Through each end of the lacing angles, in the two bottom stories of the tallest towers, pass four $\frac{7}{8}$-in. rivets, while only three $\frac{7}{8}$-in. rivets have been used for the three top stories. The bottom struts are deep angle struts, latticed on all four

sides, and the higher bents are bolted down at their middle to a pier. The depths of the intermediate transverse struts are 4, 6, 7 and 8 ft., respectively. They have been designed as box lattice girders with stiff web members and brackets at each end, portal fashion. Each chord of these struts is composed of two channels, and their ends, as well as those of the headers, are shaped fork-like so that the columns can pass between the channels of the struts and the web plates of the headers, by which means substantial and very rigid connections are obtained. The single brackets, the legs of which have an average length of 12 ft., have solid webs, and where they connect to the columns, the plates of the latter are stiffened by angles. The longitudinal diagonals form a double system, and are built of two latticed channels connected with plates at their intersection points, and where these diagonals connect with the columns the latter are provided with two diaphragms, about 2 ft. apart. Each tower has two longitudinal bottom struts, but no other longitudinal struts, which make these systems complete without the tower stringers.

The tower legs are bolted to the masonry by two 1½-in. bolts, which had been used in the old viaduct, and which vary in length from 6 ft. to about 10 ft. Provision to meet the effects of changes in temperature is made by pockets, riveted to the headers, in which the ends of the intermediate spans can slide. There are in all twelve freely moving girder ends in the viaduct, which are obtained in the following manner: Each end span is free to move, as also the intermediate span between the two smallest towers at each end of the viaduct, and five pairs of towers are united by having their intermediate spans riveted tightly between them, and, likewise, two groups of three towers each are riveted together. In this way the stresses arising from the braking of trains are reduced. The columns of the three smallest towers at each end of the structure are furnished with sliding plates, while the others are provided with nests of friction rollers.

The fixed ends of the tower legs are placed on the north side of the viaduct, as the wind blows chiefly from that direction.

The spans, Fig. 3, are plate-girder deck spans, 9 ft. on centers, with the tower spans riveted between the headers. The depth of the tower spans is 4 ft. 6 ins., and the length 38 ft. 6 ins. on centers, and for the intermediate spans the depth is 6 ft. 6 ins. and the length 61 ft. on centers. The chords of the stringers are made up of two 6 x 6-in.



ELEVATION OF HIGHEST BENT KINZUA VIADUCT,

FIG. 2.

angles and one coverplate 14 ins. wide, which runs the full length, both top and bottom, of all stringers. The spans are provided with stiff top and vertical bracings, but bottom bracings are omitted. Other dimensions and sizes, together with the principal details, are shown by the illustrations, so that it will not be necessary to refer to them.

The maximum shearing force in the webs of the plate girders is 4 000 lbs. per square inch cross-section; the maximum flange stress in the intermediate girders equals 12 000 lbs. per square inch, and that of the tower girders 11 600 lbs. The maximum stress in the towers from the supposed combinations of loading does not exceed 14 000 lbs. per square inch, while the elastic limit of the steel used is between 33 000 and 35 000 lbs.

Under the assumed loadings the anchor bolts are never called into action. The shipping weight of the deck spans complete is 638 tons and that of the towers 2 715 tons, making the total weight of the viaduct 3 353 tons. The structure was put into service about the middle of September, 1900.

Erection.

The work of erecting the viaduct has been carried on simultaneously from each end by two travelers. These carried the hoisting machinery and spanned a clear space of 160 ft., having an old tower in their middle. In this position the old tower has been taken apart, its material hoisted up and loaded on cars to be run out to the shore end of the structure, whereupon a new tower could be assembled. The work of removing the old viaduct and erecting the new one consumed four months, with a force of about 120 men at 10 hours a day.

Specifications.

The viaduct was built under the regular Erie Railroad specifications for bridges, with certain modifications as to allowable stresses, as follows: Unit stresses for live and dead loads were increased 25%, and in the flanges of the girders the whole cross-section was considered as available, but the webs of girders are spliced for flange stress, although not considered in flange section.

Where combinations of wind stresses—bending and direct—with dead-load stresses, or combinations of all the above with live-load

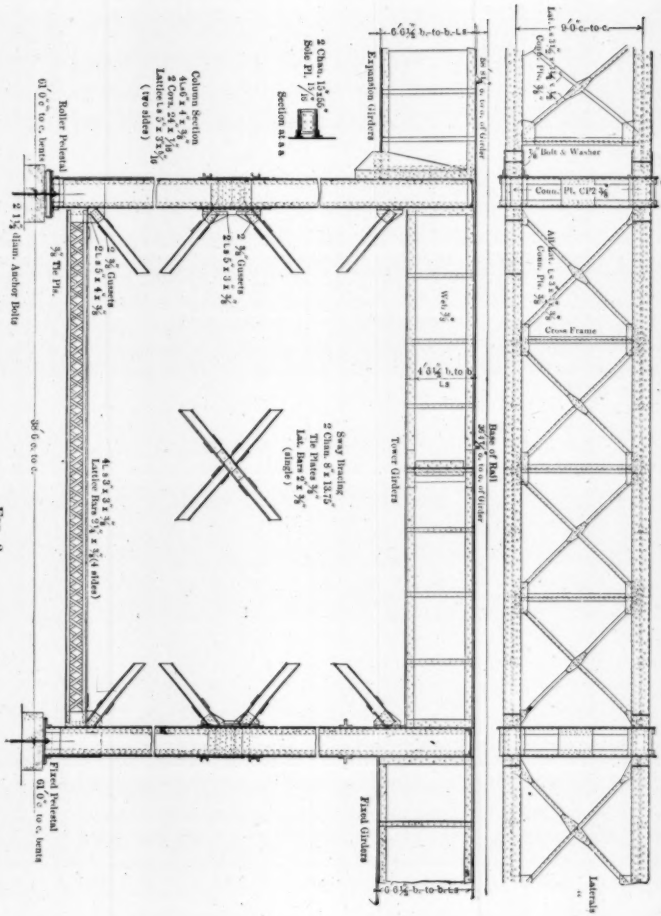

Expansion Girders
Tower Girders
Fixed Girders
Base of Rail
Section at a a
Column Section
Roller Pedestal
Fixed Pedestal
Cross Frame
Bolt & Washer
Conn. Pl. CP2
Sway Bracing
Lat. Bars 2" x 3/8"
(single)
Lattice Bars 2 1/4" x 3/8" (4 sides)
Lateral 2nd Panel
3rd "

FIG. 3.

stresses, or combinations of dead-load stresses with live-load stresses, and traction stresses from the live load exist, the stresses were not allowed to exceed the foregoing by more than 50 per cent.

The wind pressure for the loaded structure was considered to be 100 lbs. per vertical foot of bent plus 30 lbs. per square foot on floor and train. That for the light structure was considered to be 160 lbs. per vertical foot of bent plus 50 lbs. per square foot on floor.

Method of Calculating.

There is no group of stresses which calls for any special mention, except that which is due to the wind forces, as these latter are resisted by bending as well as by direct stresses, and for this reason this group of stresses alone will be investigated.

As far as known, the stresses in frames of the kind under consideration have never before been analytically discussed, and it may be noted that, in order to effect a solution, the problem can be approached from different points of view. The writer preferred a modern analytical method, proceeding from the principle of least work, but, before explaining the method of application of this principle, some general remarks in regard to these tower bents will be given.

In computing the stresses in a bent it is customary to proceed from the supposition that it acts like a cantilever, fixed at the bottom and otherwise free to move; but, while the influence of a force that acts on a bent with articulated joints is confined to a field extending from the point of application of the force to the bottom of the bent, a force that acts on a bent with fixed joints, no matter where its point of application, exercises an influence over the entire frame. A bent of only one story, fixed at the bottom and with fixed joints, will be considered first.

Such a frame partakes of the nature of a hingeless arch, and if quite a general case of loading be assumed, then this frame, like the hingeless arch, is threefold statically indeterminate; that is to say, the values of three unknown quantities must first be found before the stresses can be computed. For this supposed condition of loading none of the points of contraflexure is at the center of the different members, and those of the columns are besides in different horizontal planes. But this state of things is changed, if we have to deal with special loadings, as, for instance, the wind forces, which are supposed to act as concentrated loads at the panel points and in horizontal

FIG. 1.—NEW KINZUA VIADUCT IN PROCESS OF ERECTION.

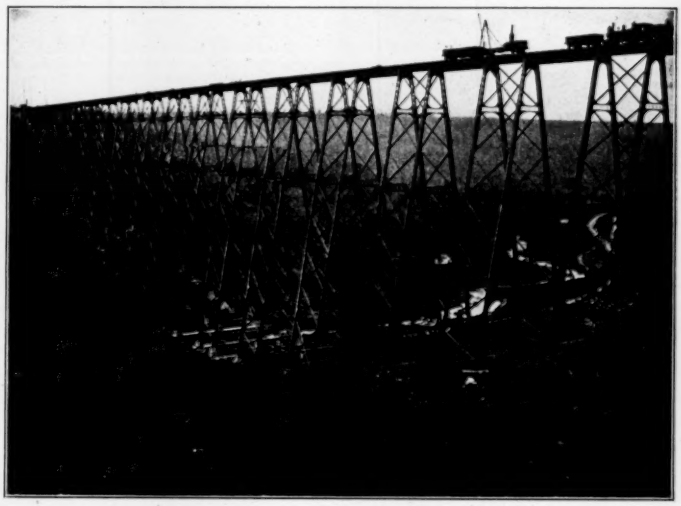


FIG. 2.—THE NEW KINZUA VIADUCT.

planes. In this case the frame becomes singly statically indeterminate. The bending moments of the columns are alike, and the points of contraflexure are on the same level, but not in the middle of these members, which points to the fact that the direct stress in the header equals half of the exterior force, and with the point of contraflexure at the center of the header.

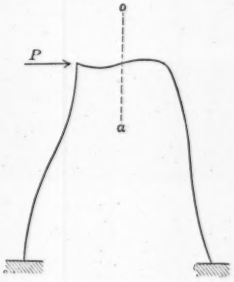


FIG. 4.

In order to illustrate the foregoing statements we pass a cut $\alpha\ \alpha$ through the center of the bent, as indicated in Fig. 4, and apply at this point the unknown vertical force X, Fig. 5, acting upward in the left fragment and downward in the right fragment of the frame. We also apply $\frac{P}{2}$ to each fragment, but a moment need not be applied at the center, because no moment can pass that point. The problem is now to determine the value of the unknown X. In like manner, in a two-story bent the direct stress in the header and in the intermediate strut is equal to half of their exterior concentrations, and their points of contraflexure are in the middle, while the bending moments of corresponding columns are alike and their points of contraflexure are not in the middle. The number of unknowns in a two-story bent is therefore 2, and in a bent of n stories the number is n.

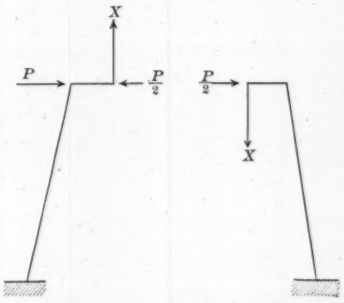


FIG. 5.

The problem can also be stated to consist of finding the locations of the points of contraflexure, because, if these locations were known, it would only be necessary to resolve the resultant of the exterior forces into the reactions, which must pass through the points of contraflexure, and, with the reactions known, the direct stress in any member and the bending moment in any point of the frame is easily computed.

Aside from the margin of safety, provided for by the specifications,

judging solely from his previous experience with frames of a similar nature, the writer felt satisfied that this graphical method would answer all practical purposes. In fact, he planned it for practical use, and only his desire to know how much the results thus obtained would vary from the analytical results prompted him to develop an analytical treatment.

The writer made the calculations for a wind pressure of 30 lbs. per square foot on girders and train and 100 lbs. per vertical foot of bent, and for five different bents, varying in height by a regular story of 62 ft., excepting the top story, which has a height of about 31 ft., and the results thus obtained answer perfectly well for the bents in the entire structure. Considering the magnitude of the stresses thus computed, an approximate determination of the effects of a wind force of 50 lbs. per square foot on the girders and 160 lbs. per vertical foot of bent appeared to be justifiable. After these few remarks the application of the principle of least work to the bents will now be shown, but, in doing so, the reader's familiarity with this principle must be assumed, as a demonstration of its truth is beyond the scope of this paper.

The bents are considered as being under no initial stress. Neglect the small alterations in the lengths of the members and consider bending alone. The principle of least work requires that the work of deformation shall be a minimum, which is expressed by

$$W = \int \frac{M^2}{2\,E\,I}\,dx = \text{Min.}$$

In this term W = work of deformation, M = moment of the exterior forces at any point, E = modulus of elasticity, supposed to be constant throughout the frame, and I = constant moment of inertia of a member under consideration. Now, if the work of deformation be a minimum, its partial derivatives, taken successively with respect to the unknowns $X_1, X_2 \ldots\ldots X_n$, treated as independent of each other, must be put each equal to zero. The equations of condition are then as follows:

$$\left.\begin{aligned}
\frac{\delta\,W}{\delta\,X_1} &= \frac{\delta\left[\int \frac{M^2}{2\,E\,I}\,dx\right]}{\delta\,X_1} = \int \frac{M}{E\,I}\,\frac{\delta\,M}{\delta\,X_1}\,dx = 0 \\
\frac{\delta\,W}{\delta\,X_2} &= \frac{\delta\left[\int \frac{M^2}{2\,E\,I}\,dx\right]}{\delta\,X_2} = \int \frac{M}{E\,I}\,\frac{\delta\,M}{\delta\,X_2}\,dx = 0 \\
\frac{\delta\,W}{\delta\,X_n} &= \frac{\delta\left[\int \frac{M^2}{2\,E\,I}\,dx\right]}{\delta\,X_n} = \int \frac{M}{E\,I}\,\frac{\delta\,M}{\delta\,X_n}\,dx = 0
\end{aligned}\right\} \ldots\ldots A$$

FIG. 1.—KINZUA VIADUCT.—CENTRAL VIEW THROUGH TOP STORY.

FIG. 2.—KINZUA VIADUCT.—CENTRAL VIEW THROUGH AN INTERMEDIATE STORY.

The equations "A" are directly applicable to the bents, but it must be noted that the integrations have to be extended over the whole frame, which is equivalent to saying that the integrals, after they have been computed with respect to the individual members of the frame, must all be added together. The reader is now referred to the stress sheet, Fig. 9, and to the table on pages 34 and 35, which the writer prepared for the sake of convenience. The stress sheets and the table contain all the data necessary for the computations, and by the use of the table the different sets of the equations of condition can readily be written down at once; but it should be remarked that each computed integral, as given in the table, must be divided by its corresponding moment of inertia and also by E, the modulus of elasticity. E, being a constant, drops out of the equations, if these are all multiplied by E.

Returning to the one-story bent and referring to the stress sheets, the method of computation is now as follows:

Header:

$$M = -X_5 r; \quad \frac{\delta M}{\delta X_5} = -r.$$

$$\int \frac{M \delta M}{\delta X_5} d x = \int_0^{h = 4.765} X_5 r^2 d r = 36.06 X_5.$$

Column:

$$M = P_5 z - X_5 (h + n z); \quad \frac{\delta M}{\delta X_5} = -(h + n z)$$

$$\int \frac{M \delta M}{\delta X_5} d x = -\int_0^{a = 31.16} [P_5 z - X_5 (h + n z)] (h + n z) d z =$$
$$- 26760.27 + 1758.54 X_5$$

The equation of condition becomes now:

$$\frac{36.06 X_5}{107500} - \frac{26760.27 - 1758.54 X_5}{10750} = 0$$

and its solution gives: $X_5 = 15.2$ tons. Likewise, the two equations of condition for a two-story bent are found from the table, viz.:

$$\frac{-26760.27 + 1758.54 X_5 - 451582.21 + 14735.11 [X_5 + X_4]}{10750} + \frac{36.06 X_5}{107500} = 0.$$

$$\frac{-451582.21 + 14735.11 [X_5 + X_4]}{10750} + \frac{329.35 X_4}{5511} = 0$$

The values of the unknowns are: $X_5 = 19.27$ tons and $X_4 = 10.9$ tons.

After the unknowns have been determined, it is then easy to calculate the bending moments at every point of the frame.

The direct stresses in the columns $= \mp \sin. \alpha \Sigma P \mp \cos. \alpha \Sigma X$, and they are tension on the windward and compression on the leeward side of the bent, α being the angle formed by the axis of the tower leg and the vertical. The shear on the columns is principally and effectively resisted by the lacing angles.

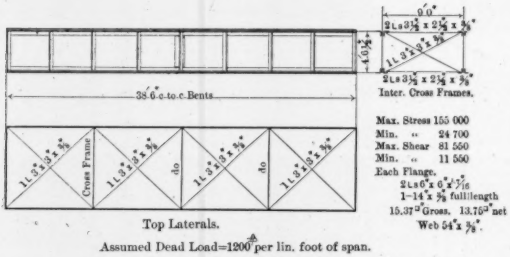


FIG. 6.

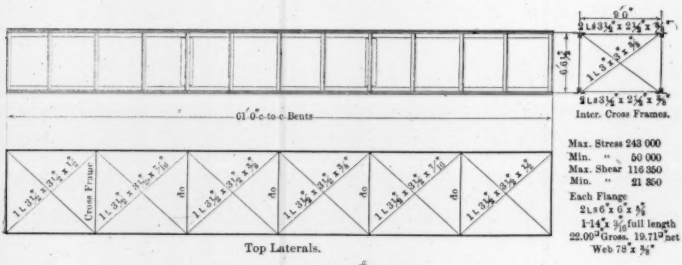


FIG. 7.

Bent of 1 story.....$X_5 = 15.2$.
" " 2 stories.....$X_5 = 19.27$; $X_4 = 10.9$.
" " 3 " .....$X_5 = 19.41$; $X_4 = 11.64$; $X_3 = 7.77$.
" " 4 " .....$X_5 = 19.43$; $X_4 = 11.67$; $X_3 = 8.10$; $X_2 = 5.86$.
" " 5 " .....$X_5 = 19.44$; $X_4 = 11.68$; $X_3 = 8.14$; $X_2 = 6.2$; $X_1 = 4.96$.

In the manner just outlined the computations for the five different bents have been made, and, although it was somewhat tedious, it in no wise proved to be a hardship. The structure of the equations of condition is such that those for a four and five-story bent have been effected by ordinary means, instead of having recourse to the theory of determinants.

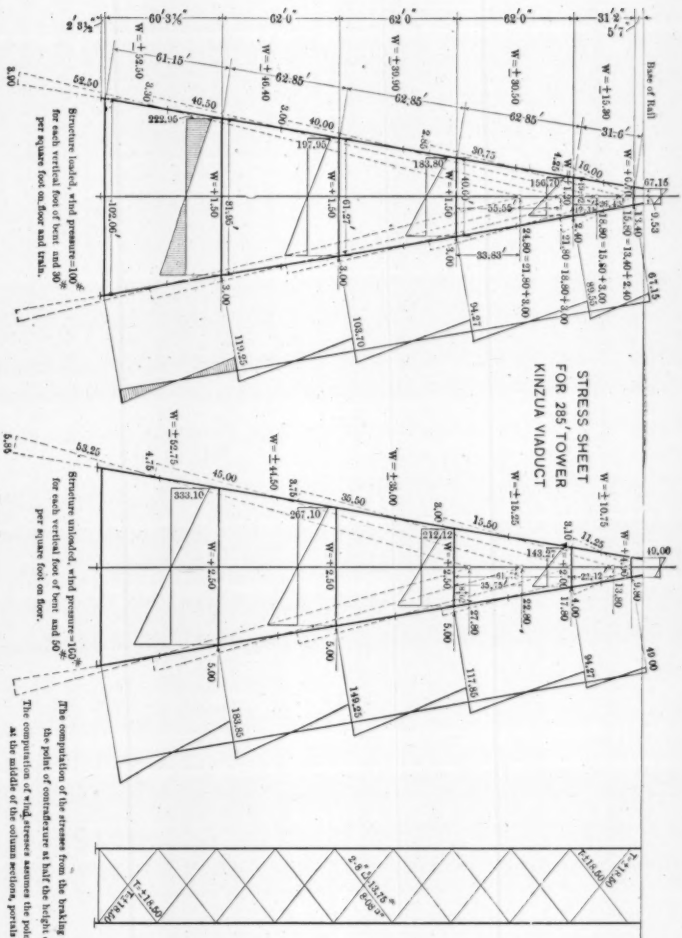
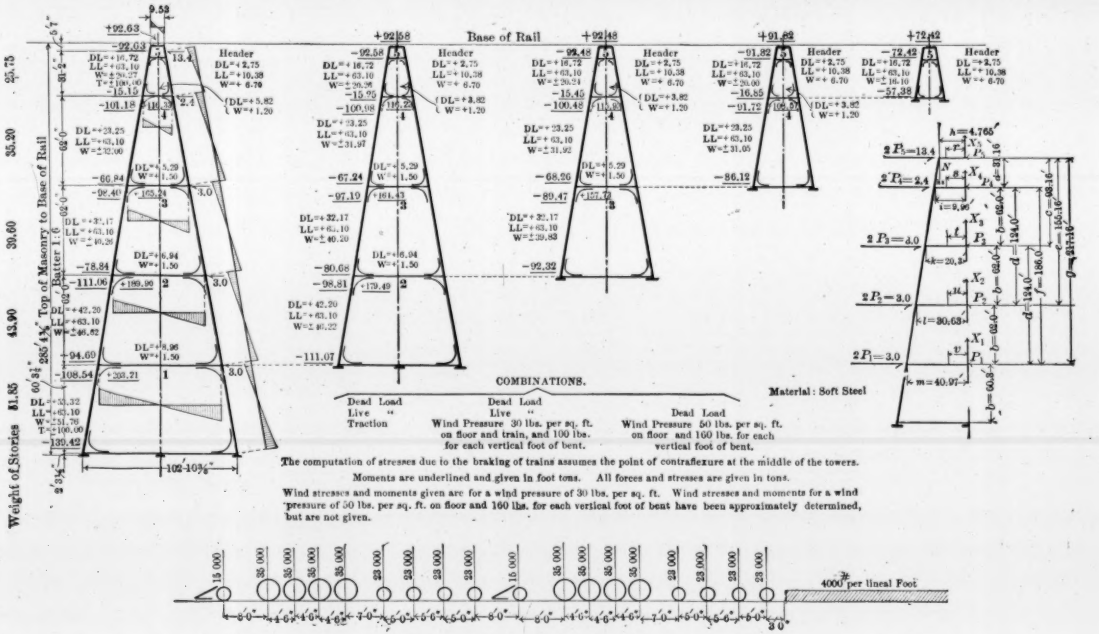

STRESS SHEET
FOR 285' TOWER
KINZUA VIADUCT
Base of Rail
Structure loaded, wind pressure=100# for each vertical foot of bent and 30# per square foot on floor and train.
Structure unloaded, wind pressure=100# for each vertical foot of bent and 50# per square foot on floor.
The computation of the stresses from the braking of trains assumes the point of contraflexure at half the height of the towers.
The computation of wind stresses assumes the point of contraflexure at the middle of the column sections, portals and headers.

FIG. 8.

Sizes.	1st-story column.	Header.	Strut "4." Twin latt. girder.	Strut "3." Twin latt. girder.	Strut "2." Twin latt. girder.	Strut "1." Twin latt. girder.
	4 ⌞'s 6 x 4 x $\frac{7}{16}$ in. 2 Pl. 24 x $\frac{1}{2}$ in. Latt. ⌞'s 5 x 3 x $\frac{5}{16}$ in.	2 ⌞'s 5 x $3\frac{1}{2}$ x $\frac{3}{8}$ in. 2 ⌞'s 6 x 6 x $\frac{1}{2}$ in. 2 Pl. $10\frac{5}{8}$ x $\frac{3}{8}$ in.	4 6″ U's 10.5 lb. Latt. ⌞'s $2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{5}{16}$ in. Latt. bars 4 x $\frac{3}{8}$ in.	4 8″ U's 13.75 lb. Latt. ⌞'s $2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{5}{16}$ in. Latt. bars 4 x $\frac{3}{8}$ in.	4 10″ U's 20.0 lb. Latt. ⌞'s $3\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{5}{16}$ in. Latt. bars 4 x $\frac{3}{8}$ in.	4 10″ U's 20.0 lb. Latt. ⌞'s $3\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{5}{16}$ in. Latt bars 4 x $\frac{3}{8}$ in.
Width and depth out to out	2′ 0½″ x 3′ 2″	2′ 1¼″ x 8′ 10″	2′ $4\frac{5}{8}$″ x 4′ 0″	2′ 5¼″ x 6′ 0″	2′ 6″ x 7′ 0″	2′ 6″ x 8′ 0″
Moment of inertia	12 356	107 500	5 511	16 692	32 514	43 803
Unknowns		X_5	X_4	X_3	X_2	X_1
Moment M of exterior forces	$P_5 g + P_4 f + P_3 d + P_2 b (P_5 + P_4 + P_3 + P_2 + P_1) z - (X_5 + X_4 + X_3 + X_2 + X_1)(m + n z)$	$- X_5 r$	$- X_4 s$	$- X_3 t$	$- X_2 u$	$- X_1 v$
$\frac{\delta M}{\delta X_5}$	$- (m + n z)$	$- r$				
$\frac{\delta M}{\delta X_4}$	$- (m + n z)$		$- s$			
$\frac{\delta M}{\delta X_3}$	$- (m + n z)$			$- t$		
$\frac{\delta M}{\delta X_2}$	$- (m + n z)$				$- u$	
$\frac{\delta M}{\delta X_1}$	$- (m + n z)$					$- v$
$\int M \frac{\delta M}{\delta X_5} dx$	$- 6502866.84 + 128073.93 (X_5 + X_4 + X_3 + X_2 + X_1)$	$+ 36.06 X_5$				
$\int M \frac{\delta M}{\delta X_4} dx$	$- 6502866.84 + 128073.93 (X_5 + X_4 + X_3 + X_2 + X_1)$		$+ 329.35 X_4$			
$\int M \frac{\delta M}{\delta X_3} dx$	$- 6502866.84 + 128073.93 (X_5 + X_4 + X_3 + X_2 + X_1)$			$+ 2788.47 X_3$		
$\int M \frac{\delta M}{\delta X_2} dx$	$- 6502866.84 + 128073.93 (X_5 + X_4 + X_3 + X_2 + X_1)$				$+ 9579.00 X_2$	
$\int M \frac{\delta M}{\delta X_1} dx$	$- 6502866.84 + 128073.93 (X_5 + X_4 + X_3 + X_2 + X_1)$					$+ 22923.27 X_1$

Sizes.	5th-story column.	4th-story column.	3d-story column.	2d-story column.
	4 ⌊'s 6 x 4 x $\frac{3}{8}$ in. 2 Pl. 24 x $\frac{7}{16}$ in. Latt. ⌊'s 5 x 3 x $\frac{5}{16}$ in.	4 ⌊'s 6 x 4 x $\frac{3}{8}$ in. 2 Pl. 24 x $\frac{7}{16}$ in. Latt. ⌊'s 5 x 3 x $\frac{5}{16}$ in.	4 ⌊'s 6 x 4 x $\frac{3}{8}$ in. 2 Pl. 24 x $\frac{7}{16}$ in. Latt. ⌊'s 5 x 3 x $\frac{5}{16}$ in.	4 ⌊'s 6 x 4 x $\frac{7}{16}$ in. 2 Pl. 24 x $\frac{1}{2}$ in. Latt. ⌊'s 5 x 3 x $\frac{5}{16}$ in.
Width and depth out to out	2′ 0½″ x 3′ 0⅞″	2′ 0½″ x 3′ 0⅞″	2′ 0½″ x 3′ 0⅞″	2′ 0½″ x 3′ 1″
Moment of inertia	10 750	10 750	10 750	12 356
Unknowns				
Moment M of exterior forces	$P_5 z - X_5 (h + n z)$	$P_5 a + (P_5 + P_4) z - (X_5 + X_4)(i + n z)$	$P_5 c + P_4 b + (P_5 + P_4 + P_3) z - (X_5 + X_4 + X_3)(k + n z)$	$P_5 e + P_4 d + P_3 b + (P_5 + P_4 + P_3 + P_2) z - (X_5 + X_4 + X_3 + X_2)(l + n z)$
$\frac{\delta M}{\delta X_5}$	$-(h + n z)$	$-(i + n z)$	$-(k + n z)$	$-(l + n z)$
$\frac{\delta M}{\delta X_4}$		$-(i + n z)$	$-(k + n z)$	$-(l + n z)$
$\frac{\delta M}{\delta X_3}$			$-(k + n z)$	$-(l + n z)$
$\frac{\delta M}{\delta X_2}$				$-(l + n z)$
$\frac{\delta M}{\delta X_1}$				
$\int M \frac{\delta M}{\delta X_5} dx$	$-23760.27 + 1758.54 X_5$	$-451582.21 + 14735.11 (X_5 + X_4)$	$-1594211.87 + 40761.72 (X_5 + X_4 + X_3)$	$-3629878.5 + 79998.14 (X_5 + X_4 + X_3 + X_2)$
$\int M \frac{\delta M}{\delta X_4} dx$		$-451582.21 + 14735.11 (X_5 + X_4)$	$-1594211.87 + 40761.72 (X_5 + X_4 + X_3)$	$-3629878.5 + 79998.14 (X_5 + X_4 + X_3 + X_2)$
$\int M \frac{\delta M}{\delta X_3} dx$			$-1594211.87 + 40761.72 (X_5 + X_4 + X_3)$	$-3629878.5 + 79998.14 (X_5 + X_4 + X_3 + X_2)$
$\int M \frac{\delta M}{\delta X_2} dx$				$-3629878.5 + 79998.14 (X_5 + X_4 + X_3 + X_2)$
$\int M \frac{\delta M}{\delta X_1} dx$				



The computation of stresses due to the braking of trains assumes the point of contraflexure at the middle of the towers.

Moments are underlined and given in foot tons. All forces and stresses are given in tons.

Wind stresses and moments given are for a wind pressure of 30 lbs. per sq. ft. Wind stresses and moments for a wind pressure of 50 lbs. per sq. ft. on floor and 160 lbs. for each vertical foot of bent have been approximately determined, but are not given.

STRESS SHEET; KINZUA VIADUCT.

FIG. 9.

The stress sheets show also the direct stresses and bending moments of the highest bent from the wind forces found by graphic statics as explained before, under the assumption that the points of contraflexure, through which the reactions must pass, are in the middle of each member of the bent.

The forces are all given in tons and the moments in foot-tons. The algebraic sum of the moments around a panel point must equal zero. Roughly speaking, the moments of the struts are about twice as great as those of the columns.

The author made a special investigation, which brought out the fact that the moments of inertia of a group of members may be changed as much as 100% without materially altering the final results.

DISCUSSION.

Mr. Morison. GEORGE S. MORISON, Past-President, Am. Soc. C. E. (by letter).—The structure by which the old Kinzua Viaduct has been replaced, while containing much of the modern practice of detail connections and riveting, differs from other viaducts in two radical features. The first of these lies in the fact that the tops of the two battered posts of each bent are brought so near together, and are so rigidly connected, that the bent instead of being a quadrilateral is treated as an isosceles triangle. The second peculiarity is that all transverse diagonal bracing is omitted, the posts being stiffened only by their own unusually large transverse dimensions, assisted by the cross-struts and the rigidity of the connections between those cross-struts and the posts.

The first of these features is to be commended. The ideal trestle bent would be an isosceles triangle carrying a superstructure so narrow that the weight is all taken at a single point on top. While this cannot be done, the design of the new Kinzua Viaduct approaches sufficiently near to it for practical purposes. With bents of such moderate height that intermediate stiffening of the posts becomes unnecessary, this form of construction seems all that can be desired. There are, however, two very important limitations: The first of these is in height, new complications being introduced whenever transverse stiffeners are required; and the second is that it is only applicable to single-track structures, the eccentric loading of a double-track bridge producing irregular conditions at the top of the structure.

The second peculiar feature is of a more radical type. In the usual design each bent becomes a cantilever truss, standing vertically instead of horizontally; anchored, by weight or otherwise, to the masonry foundations, resisting a horizontal instead of a vertical load, and with its stiffness determined by the unit strains in the two chords and the distance between those chords. In the Kinzua design the cantilever form is retained, but the cantilever, instead of resisting horizontal forces for its whole length, resists them only at the top. The struts at the intermediate points are simply stiffeners, their only function being to control the curves of flexure of the posts. This flexure is naturally due to two causes: The first is wind; the second is the usual perversity of all compression members in buckling under loads, which perversity increases with the length of the member. Were the transverse struts connected to the posts by pins, their effect as stiffeners would be comparatively small, as the transverse deflections of the two posts would be represented by approximately parallel curves, the only effect of the stiffeners being to maintain the parallelism of these curves. Such struts would undoubtedly increase the

stiffness of the posts, considered simply as compression members, but would add little to their ability to resist wind. In this case, however, the struts are rigidly attached to the posts, so that the angle made by the strut and the post must always remain constant. This introduces new conditions. If the struts were absolutely stiff (an impossible supposition) the direction of the posts at their points of intersection would always remain unchanged, but the posts of the lower part of each panel would bend from the wind in a curve the center of which is on the leeward side, recovering their direction in the upper part of each panel by a curve the center of which is on the windward side. As a moderate amount of flexure takes place also in the transverse struts, the actual bending in the lower part of the panel exceeds the recovery in the upper part of the panel. The result, on either supposition, is a series of cumulative deflections. The top of the lowest panel is deflected by an amount determined by the wind pressure and the stiffness of the posts. The second panel has its own similar deflection, but, as the base of the second panel is the top of the lower panel, the deflection of the top of the second panel is the sum of the two deflections; and so on to the top of the tower. The principle on which this structure appears to have been designed is that these various bendings of the posts shall produce no strains beyond safe limits in the extreme fibers of those posts. It involves calculations of a complicated and abstruse character, but it is possible to design a structure in this way which is perfectly secure. Mr. Morison.

The frames on many tall office buildings have been designed on this principle, though in many instances the calculations have been of the most crude and careless kind. The portal of a through bridge is really a construction of the same order. This principle, however, has generally been used only in those places where there were serious objections to transverse diagonal bracing. The general practice indicates that there are good reasons why engineers have always preferred diagonal bracing when there were opportunities for putting it in. These reasons are of two kinds: First, with diagonal bracing and a base of sufficient width to prevent tension in anchorages, there is never any reversal of strains in the posts, and, furthermore, the strains caused by wind alone when distributed over the whole cross-section of the post produce very small unit stresses and very slight changes in lengths of posts. In the present design reversals of strains are likely to occur in the extreme fibers of almost every part of each post, and these involve an appreciable change in the form of the post. The second reason is that the stiffness of a beam or truss is in direct proportion to its depth, and, with the opportunity of giving a trestle bent a stiffness based on the whole width of its base, it does not seem right to accept a stiffness based only on the width of the post.

Bridge specifications, many of which are drawn with great care,

Mr. Morison. generally have the defect of specifying strength and not providing for stiffness. Under such specifications two structures can be accepted as equally good, though, under a load, one of them will have twice the distortion that the other will have. For immediate safety, strength is all that is needed; for long life of structure, stiffness is at least equally important. This objection may be raised to some special forms of trusses, favored because of their economy of material, in which much greater variations occur in the strains of individual members than in differently designed trusses containing a little more metal. As compared with a viaduct of the usual design, everything would seem to indicate that the transverse deflections under strain, and even the distortion of the posts under compression, would be large in the new Kinzua Viaduct. If this be so, it is not probable that this design will be repeated. The objection to diagonal rods in the panels of such a viaduct, with pin and turn-buckle connections, is perhaps more theoretical than real, and, in a structure with the rigid and excellent details of the one described, they certainly could do no harm. It would be an interesting thing to put such rods in one of the tall towers of this viaduct, and then compare the stiffness of this tower and the adjoining tower whenever an extraordinary storm passes through the valley.

Mr. Buck. R. S. Buck, M. Am. Soc. C. E.—The Kinzua Viaduct is one of the best known and most unique railroad bridges in America, and its reconstruction will doubtless command general attention, both at home and abroad. This paper and the articles appearing in the leading engineering journals, while not furnishing data adequate to form a conclusive verdict as to the merits or demerits of the design, furnish sufficient ground for discussion of certain of its salient features. It is quite possible that a more thorough treatment of the subject by those fully acquainted with the design may disarm some of the following adverse criticisms; but the speaker believes that the showing on which he is compelled to base his discussion fully justifies its character.

The author does not, in the speaker's opinion, furnish such full explanation of his methods of calculation, nor as many details, as the radical character of the design demands.

The paper leaves so much to the ingenuity and imagination of the reader that it is as difficult to reach a full understanding of his methods and deductions as it is to find full justification for the design. The calculations are involved, and appear to be contradictory. The reader is subjected to much unnecessary trouble by the absence of a clear statement of the meaning of terms in the equations, limits of integration, and the steps by which the several expressions on pages 34 and 35 are reached.

The speaker has been unable, despite persistent effort and some able outside assistance, to follow the calculations to a finish, but some points stand out in a measure independent of the statement of the general scheme, and can be judged as correct or otherwise.

Mr. Buck.

On page 29 appears the statement:

"The problem can also be stated to consist of finding the locations of the points of contraflexure, because, if these locations were known, it would only be necessary to resolve the resultant," etc.

This refers to the analytical treatment, but, beyond this statement, there is nothing discernible in the text, table of equations or stress diagrams, to indicate that any effort has been made by the author to find the true points of contraflexure or to place them anywhere else than arbitrarily at the centers of the members.

Referring to the graphical treatment on the stress diagram, Fig. 8, is the note: "The computation of wind stresses assumes the point of contraflexure at the middle of the column sections, portals and headers."

These two quoted statements conflict, and should be further explained, for, until they are, there is no logical sequence in what follows. It is quite possible that the points of contraflexure are not very far from the centers of the members in all cases except in the case of the bottom sections of the columns. Here, the assumption is materially erroneous, and the effect on the moments very considerable.

In this case, the author not only assumes the column to be absolutely fixed at its foot, but also apparently ignores the bottom strut as resisting bending at this point. As a matter of fact, the fixity of the column at its foot is almost entirely dependent upon this strut.

The only other features that tend to fix it are the two 1½-in. anchor bolts very near the center of the base, and the direct pressure on the column acting on its square end.

The anchor bolts, intended to resist only 12 000 lbs. direct tension, can do but little to resist bending in the column. With rollers under the leeward column and with direct pressure removed by up-lift on the windward column, due to the assumed wind load on the unloaded structure, the fixity of the columns due to direct weight is very uncertain.

In the opinion of the speaker, the conditions do not justify the assumption of more fixity at the bottom of the columns than that furnished by the bottom strut. The correct location of the point of contraflexure of the bottom section is not far above the foot of the column, instead of at the center. The result of this is to greatly increase the bending at the level of Strut 1 over what is shown in the author's results, and also to increase the up-lift on the windward column enough to make it exceed the dead-load stress, with the assumed wind load on the unloaded structure.

The speaker would like to know if the author means, by the closing paragraph of his paper, that if the moments of inertia of a group of members were doubled it would not materially increase the stiffness of the bent? Also, how many members he considers as "a group"?

Mr. Buck. The failure of the author to give the essential details of connections bearing on the peculiar features of this design renders it necessary, in order to even approach an adequate understanding of the subject, to have recourse to the articles on the same subject in the technical journals.* These articles appear to be based on data from an authentic source, and to bear the stamp of official sanction. It is therefore deemed proper to refer to these articles for information essential to an adequate discussion of the subject, but lacking in the paper. It is to be hoped that the author will furnish all these details, in order to render the paper complete within itself and to correct any error which may arise from having to go outside of the paper for information.

Assuming that the author's methods of calculation are reasonably approximate—which does not seem warranted until they are rendered logical and free from obscurity—there are still, in the opinion of the speaker, faults in the design. He believes it is a mistake to reject the time-honored diagonal bracing in a structure of such proportions, and to substitute a form of bracing which, even when most carefully and generously detailed, could hardly be made both sure and economical. In all bridges, outside of a suspension bridge, rigidity is one of the prime objects to be attained, and especially in the case of railroad bridges. Substituting for direct stresses bending in members, being themselves certainly not possessed of a high degree of rigidity, is depriving the concrete structure of rigidity to an extent that seems questionable.

The height of this viaduct, at its highest point, is 285 ft. from masonry to base of rail. Each tower is independent, and can receive its maximum wind stress with wind over only about 150 ft. longitudinally of the structure. Within this narrow limit wind strikes in gusts and with considerable impact. In a tower so high, where 54% of the wind pressure is applied at the top when loaded, and 34% when unloaded, where the dead load is not sufficient to prevent up-lift on the windward column with assumed wind on the unloaded structure, the dynamic effect of vibration is a consideration that should be provided for. In a design of this kind, the amount of lateral deflection of the towers under wind load should be carefully ascertained, and, although the speaker has made no calculation for this purpose, he deems it safe to venture the statement that it is much greater in this design than in the case where the ordinary form of bracing is used.

It would be a matter of great interest to know, from transit observations, just what the present structure deflects laterally under high wind.

The detailing of the lateral strut connections with the columns seems to the speaker hardly in keeping with the heroic and uncertain

* *Railroad Gazette*, Nov. 30th, 1900, and *The Engineering Record*, Dec. 1st, 1900.

service they have to perform. The use of clip angles to reinforce the attachment of the strut chords to the columns, applied so as to be almost wholly ineffectual, is not warranted by what appears to be the only consideration for so doing, namely, the saving of some weight in the gusset plates at these points. Mr. Buck.

The light character of the knee braces, and their method of attachment to the struts, seems to indicate that their function is ornamental rather than useful; yet, with sufficient metal and adequate attachment to the columns and struts, they could be made to add very materially to the strength and rigidity of the structure.

A study of these details goes far to explain the material saving in weight with which the author credits this design, for they bear evidence of scaling down the metal to an extent that can hardly be generally endorsed.

There is another feature of this design which, independent of the character of the lateral bracing, seems to lack due strength, namely, the 1½-in. anchor bolts which, having served in the old structure, now serve in the new. In the old structure it is understood that there were no rollers under either column, temperature movements being taken care of by slotting transversely the anchor bolt holes in the column shoes, thus permitting the shoes to slide on top of the piers. The bottom strut was probably bolted fast to the center pier. With the bridge unloaded there was a wind pressure of about 45 000 lbs. on each tower, or 22 500 lbs. on each bent, under the assumed wind loads on the unloaded structure. To resist this shear at the top of the piers, there being up-lift on the windward column, there was friction under the leeward shoe, the attachment at the top of the center pier and the shear of probably one pair of anchor bolts in the column piers. These connections, though somewhat uncertain because of the slotted holes, were doubtless adequate, especially as the friction under the leeward shoe alone was perhaps equal to the wind shear.

In the case of the new structure, the service demanded of these anchor bolts is much more severe. The shear is 55 600 lbs. to the bent, 2½ times that in the old bridge. Rollers are placed under the column to leeward of the prevailing heavy gales, and there is therefore practically no frictional resistance at the foot of this column, and, the anchor bolt holes being slotted 6 ins. on this shoe, the bolts can take no shear from the shoe under normal conditions. It is also reasonable to assume that the anchor bolt holes in the bottom strut at the center pier were slotted. With the assumed wind load on the unloaded structure there is, as before stated, up-lift on the windward column. Therefore, with practically no friction under the shoe of this column the whole 55 600 lbs. of the wind shear has to be taken up by the two 1½-in. anchor bolts in the windward pier. This seems to be more than should be exacted of such bolts, and it is difficult to understand how

Mr. Buck. the use of the old bolts, without reinforcement, could be justified where the wind shear was so greatly increased and the means of resisting it so materially reduced. In short, the factor of safety of the anchorage of bents to piers is reduced at least 2½ times, and, in addition, by the specific amount of friction lost under the leeward shoe.

The speaker fails to see where any advantage is gained by placing the rollers on the side away from the direction of prevailing heavy winds, as the capacity of the towers to resist the shear due to such winds is thereby so greatly reduced.

The bolts in the movable shoe should be just as capable of resisting direct tension from up-lift as those in the fixed shoe, and there would be no loss of friction, but, in fact, a gain, when the fixed shoe is to leeward.

Mr. Moisseiff. L. S. MOISSEIFF, Assoc. M. Am. Soc. C. E.—This paper offers so many novel features and distinct departures from the usual design of high viaducts in the best modern American practice that the discussion of its merits or defects should prove to be of considerable interest to the profession.

While designed according to the rather conservative specifications of the Erie Railroad Company, the design of the high towers denotes a radical departure from the type used heretofore, in omitting the sway diagonals. The bending moments caused by the assumed wind loads are intended to be resisted by the increased stiffness of the columns and horizontal struts which are strengthened by brackets. Not that this method of providing for wind stresses is entirely unknown or untried; it has been used very frequently, and with some reluctance, for sway bracing of through bridges, for wind bracing of high steel-skeleton structures, and in all portals. But, in all cases where sway diagonals have been omitted, this omission has been due to lack of head room or due to wall space taken up by windows and doors, and was thus made compulsory by the necessities of the case.

It seems so much to be almost an axiom of good structural design never to resist by bending stresses what can be done as well by direct axial stress, that the application of the "portal bracing" principle in a high viaduct, subject to high wind pressures and having no limitations of head room, is surprising.

Every engineer who has had to design trusses and frames of great magnitude, and containing long members which require bracing and intermediate supports, knows the great amount of material required by these secondary members and how rapidly the percentage of material required to attain the practical design of the truss increases in comparison with the stress volume. Every attempt to reduce the cost of these secondary members is a step in the right direction; but it seems doubtful whether the saving in material claimed by the author for his design has been obtained without impairing the efficiency of

the structure to resist lateral forces and vibrations, especially the latter. Mr. Moisseiff.

While a comparison of the tower bracing as designed with the usual diagonal bracing of high trestles and towers with stiff members will certainly show a saving in weight for the portal-braced type, a comparison of their relative stiffness will be greatly in favor of the one with diagonal braces. A high trestle bent of the type used by the author is certainly not stiffer than the old tie-rod braced bent. In order to get a fair comparison between the portal-braced system and the system with diagonals, the former should be compared with a bent braced by tie rods. The speaker believes that the claimed economy will then soon vanish. The bending moments induced by wind pressures may be provided for by the stiff and deep struts just as well as by rigid diagonals, but the vibrations caused by them will be considerably higher than in diagonally braced towers. The tower will, in fact, have some similarity to a tuning fork.

The author deserves credit for the application of modern methods to the analysis of the stresses. The principle of least work used by him is a handy and convenient tool for the determination of stresses in frames, and especially for the study of new types of trusses. The value of the author's analytic treatment would be greatly enhanced if the notation used and the processes gone through were stated more explicitly and carefully.

In his analysis the author assumes each frame to be fixed rigidly at the bottom. This assumption is erroneous. The two old wrought-iron bolts, of 1½ ins. diameter and some 18 ins. apart, are not sufficient to insure the necessary degree of fixedness to the columns. Besides, the high bents have one leg on roller bearings, which is certainly not very conducive to the rigidity of the lower column end. But this assumption of fixed ends for the lower ends of the columns is a vital and extremely important factor in the author's theory. These ends are only in so far fixed as the bottom horizontal struts provide for, and not much more.

Instead of considering for a one-story bent, as the author does, the upper horizontal beam only, the correct way would be to take also the lower horizontal strut into account, and include its work of deformation in the general formula. The same idea should be followed through for all bents, always including the bottom horizontal strut as an essential member.

For a single-story bent the problem is quite simple. In view of the general application of this case for sway bracing and portals of bridges, a simple formula to find the points of contra-flexure of the columns will be given here which does not necessitate in its use the application of the higher calculus, and is rather handy.

Professor Winkler* deduces the same formula, and the following is

* "Querkonstructionen der Eisernen Brücken."

Mr. Moisseiff. a simplified deduction thereof by the principle of least work, as used by Mr. Grimm.

The work done by any beam subject to bending, in assuming its new position of equilibrium, is given by

$$A = \int \frac{M^2}{2\,E\,I}\,d\,x,$$

neglecting the work done by the axial force. E is here the modulus of elasticity, and I the moment of inertia of the beam in flexure. If the beam is fixed at both ends, Fig. 10, and is of length l, then the moment at any point x is

FIG. 10.

$$M = M_1 + (M_2 - M_1)\,\frac{x}{l}.$$

In order to find the work done by the beam in attaining its new equilibrium, we substitute this value in the foregoing work formula:

$$M^2 = M_1^2 + (M_2 - M_1)^2\,\frac{x^2}{l^2} + 2\,M_1\,(M_2 - M_1)\frac{x}{l}$$

$$A = \int \frac{M^2\,d\,x}{2\,E\,I} = \frac{1}{2\,E\,I}\int_o^l \Big[M_1^2 + (M_2 - M_1)^2\,\frac{x^2}{l^2} +$$

$$2\,M_1\,(M_2 - M_1)\,\frac{x}{l}\Big]\,d\,x = \frac{l}{6\,E\,I}\,[M_2^2 + M_1^2 + M_1\,M_2]$$

This holds true for any beam fixed at both ends by the moments M_1, M_2. This being given, the one-story bent can now be treated.

Fig 11 represents a frame of height h, and horizontal struts with fixed joints subject to the action of a horizontal force 2 P at the joint 1. Let $O\ O$ denote the line of contraflexure, and h_1 and h_2 the distances from the points of contraflexure to the upper and lower struts, respectively. The total amount of deformation work caused by 2 P consists of the sum of the work done by its four individual members.

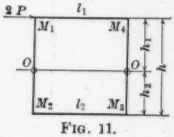


FIG. 11.

$$A = \frac{1}{6\,E}\Big[\frac{h}{I}\,(M_1^2 + M_1\,M_2 + M_2^2) + \frac{l_2}{I_2}\,(M_2^2 + M_2\,M_3 + M_3^2) +$$

$$\frac{h}{I}\,(M_3^2 + M_3\,M_4 + M_4^2) + \frac{l_1}{I_1}\,(M_4^2 + M_4\,M_1 + M_1^2)\Big]$$

where I is the moment of inertia of the columns I_1 of the upper strut, and I_2 of the lower strut. For equilibrium, the work done will be a minimum.

The moments are:

$M_1 = Ph;\ M_2 = Ph_2 = P\,(h - h_1) = Ph - M_1;\ M_3 = -\,M_2;\ M_4 = -\,M$

Differentiating partially with respect to h_1 as the independent variable:

$$\frac{\delta\,M_1}{\delta\,h_1} = P;\ \frac{\delta\,M_2}{\delta\,h_1} = -\,P;\ \frac{\delta\,M_3}{\delta\,h_1} = +\,P;\ \frac{\delta\,M_4}{\delta\,h_1} = -\,P.$$

6 E and P, as constants, divide out, and we get: Mr. Moisseiff.

$$\frac{A}{\delta h_1} = 0 = \left[\frac{h}{I}(M_1 - M_2) + \frac{l_2}{I_2}(M_3 - M_2) + \frac{h}{I}(M_3 M_4) + \frac{l_1}{I_1}(M_1 - M_4)\right]$$

Introducing the values set for the moments:

$$\frac{h}{I}(2h_1 - h) + \frac{l_2}{I_2}(h_1 - h) + \frac{l_1}{I_1}h_1 = 0,$$

whence,

$$h_1 = h\,\frac{\frac{h}{I} + \frac{l_2}{I_2}}{\frac{2h}{I} + \frac{l_2}{I_2} + \frac{l_1}{I_1}}$$

This formula gives the position of the point of contraflexure, and with it the moments at the four joints are readily found. It will be noted that the position of the points of contraflexure is independent of the external force, and that it is only a function of the moments of inertia of the members and their lengths. This might have been expected to be so.

For all the members of the same moment of inertia, and for the horizontal struts of the same lengths, $I_1 = I_2 = I$; $l_1 = l_2$, we find

$$h_1 = \frac{h}{2}$$

The point of contraflexure is at the middle of the column. Thus we find that the assumption that the point of contraflexure is at half the height of the columns is true only for a rectangle with sides having the same moment of inertia.

Applying the above contraflexure formula to the one-story bent taken by the author, the point of contraflexure will be almost 21.36 ft.

From the general equation of moments, as given by the author,

$$M = P_5 z - X_5 (h + n z).$$

For the point of contraflexure $M = P_5 h_1 - X_5 (h + n h_1) = 0$.

Whence

$$X_5 = \frac{P_5 h_1}{h + h_1 n} = \frac{6.7 \times 21.36}{4.765 + \frac{21.36}{6}} = 17.2 \text{ tons.}$$

The greatest moment at Joint 1, $M_1 = -X_5 \times 4.765 = -81.85$ foot-tons.

$$M_2 = 37.6 \text{ foot-tons.}$$

A comparison of the author's figures shows the effect of his assumption of fixed ends. The point of contraflexure will naturally be farthest from the stiffer joint, and it actually moves down and thereby increases the moment on top by some 14 per cent. Of course, the lower bents, as designed by the author, are so massive that they offer more than ample resistance to any increased moments. But this is due to the allowed low working stress.

Mr. Moisseiff. The lower horizontal strut is only 30 ins. deep, and its moment of inertia is less than one-twentieth of that of the upper 8-ft. deep strut. The point of contraflexure will, of course, move nearer to the lower end, and, making all due allowance for the stiffening effect of the brackets, it will be found to be 0.87 of the height of the lower story from the 8-ft. strut, or 52.7 ft. The sum of all the X-forces above the point of contraflexure, which is but the transferred wind load, will be 52.5 tons, and the maximum moment in the column will be 192.5 foot-tons, instead of 108.54 foot-tons as given by the author, or some 78% higher. The extreme fiber stress due to this moment in the columns is 7 000 lbs. per square inch.

Taking the stresses found by the author for dead, live, and wind load and omitting the traction stresses, and adding to them the extreme fiber stress due to bending, the greatest stress will reach 15 500 lbs. per square inch. This exceeds by 11% the maximum stress of 14 000 lbs. given by the author. Including traction, the stress will be 20 000 lbs.

It would appear from the paper that the lower bents are simply cut-off upper portions of the high bent, so that, for instance, in the four-story bent the 8-ft. horizontal strut forms the lowest member. If this is so, and if the author appreciated the great stiffening effect of the lower strut, why did he not put one as deep or deeper in the five-story bent, instead of the comparatively flimsy 30-in. strut? And if he deemed the bottom horizontal struts of no importance, as it results from his theory, then why did he not replace these deep bottom struts by the light ones used in the highest bent, and save material thereby?

Mr. LaChicotte. H. A. La Chicotte, M. Am. Soc. C. E.—The principal feature of this paper is the method of calculating the stresses in the structure, although if the title of a paper is an index of the contents, it is to the structure itself that the author would direct particular attention. That part of the paper devoted to a description and illustration of the work is so limited and devoid of detail as to necessitate going to outside sources for information relative to this structure sufficient to render an intelligent discussion of the subject possible. Fortunately, timely articles, appearing in recent issues of two technical journals, supply many of the necessary data which should have been given by the author.

The reasons offered for adopting such a design as the author submits for criticism are by no means satisfactory, and it is to be hoped that discussion will bring out the relative merits of the several designs referred to, as well as a properly executed design of the articulated type. While the latter type was excluded from competition, it would no doubt possess advantages over that used, both in economy and rigidity. It is significant that the weight of the new structure should be nearly double that of the old, even after making due allowance for the increased traffic.

Mr. LaChicotte.

The fact that the columns in a bent approach so nearly together at the top is of vital importance in the adopted design; and, without the verification of actual figures, the use of this type for a double-track structure would be precluded by reason of the greatly increased bending moments in the tower bents, and the resulting increased cost.

It is not necessary to state that the most suitable design for any structure is that which most nearly fulfills all the requirements of each particular case; and while allowance may reasonably be made for the personality of the designer, where matters of judgment and taste are involved, it is nevertheless incumbent upon him to adhere strictly to the principle of utility and suitability of design. This principle, when broadly interpreted, will be found to be amply elastic to cover all practical cases. Originality and fertility of resource are always commendable and usually valuable, but, unless guided by good judgment and practical common sense, are apt to lead to extremes. Neither prejudice against established forms, nor a liking for the unique may be allowed to dictate what a design should or should not be.

In the present case there are several details of construction open to criticism, and, while the speaker does not wish to be understood as condemning the design as a whole, there is nothing in the paper under discussion, nor in the requirements of the case, so far as known to him, which, in his opinion, would justify its adoption.

As to the analytical treatment for the determination of wind stresses, the author has presumed too much and left so much unexplained that to read the paper intelligently means to almost work out the problem on independent lines.

In fact, it was not until the speaker had worked out the problem for himself that he realized wherein lay the difficulty with the author's method.

Why the author felt justified in considering the columns as fixed at the base is past comprehension; and he is clearly in error in so doing. In the case of a one-story bent, not only is the base of the tower leg not fixed, but very much less fixed than the top, and the point of contraflexure in the column should be somewhat below the middle—that is, nearer to the smaller moment tending to fix the ends.

An incongruous result of the author's assumption and treatment is that the point of contraflexure is nearer the greater moment assumed as applied to the base of the column to insure its absolute fixity.

As a matter of fact, in applying the principle of least work, it should be made to cover the strut connecting the bases of the columns, as well as the other members of the bent, as little or no reliance can be placed in the anchor bolts as fixing the legs—especially on the expansion side of the tower.

The moments in the lower struts may always be expressed in terms of the moment of the adjacent column. If, in the expression for the

Mr. LaChicotte. moment of the column, $M = P_5 z - X_5 (h + n z)$, z is put equal to a, there results, $M_a = 208.77 - 9.96\ X_5$, for the moment in the strut when the sign of M is changed.

This moment divided by the half width of the tower base gives

$$X_4 = \frac{M}{i} = 20.96 - X_5.$$

Whence

$$\int M \frac{\delta M}{\delta X_5} dx = -\left[20.96 - X_5\right]_{x=o}^{x=9.96} \frac{x^3}{3} = -6972.69 + 332.67\ X_5.$$

This result, divided by the moment of inertia of the strut, and added to the equation of condition already found for the other members of the bent gives,

$$\frac{36.06\ X_5}{107500} - \frac{26760.27 - 1758.54\ X_5}{10750} - \frac{6972.69 - 332.67\ X_5}{5511} = 0$$

Whence, $X_5 = 16.8$, $X_4 = 20.96 - X_5 = 4.16$.

The resulting moments (Fig. 12) on the bent are:

At top of Column $M_5 = 80.1$;

At base of Column $M_4 = 41.43$.

The point of contraflexure is located 20.6 ft. from the top of the column, or, about 5 ft. from the theoretical middle point assumed in the graphical solution.

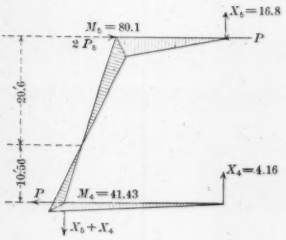


FIG. 12.

Proceeding in a similar manner in the case of a two-story bent, the unknown force acting at the middle of the strut uniting the bases of the columns is found to be

$$X_3 = 34.363 - (X_5 + X_4),$$

the equation of moments $M = -[34.363 - (X_5 + X_4)]\ x$,

from which there results:

$$\int M \frac{\delta M}{\delta X_5} dx = -\left[(34.363 - [X_5 + X_4])\right]_{x=o}^{x=20.3} \frac{x^3}{3} = -95\,820.49 + 2\,788.48\ (X_5 + X_4)$$

also,

$$\int M \frac{\delta M}{\delta X_4} dx = -\left[(34.363 - [X_5 + X_4])\right]_{x=o}^{x=20.3} \frac{x^3}{3} = -95\,820.49 + 2\,788.48\ (X_5 + X_4).$$

These expressions, when divided by the moment of inertia of the lower strut, must be added to the equations of condition already

found by the author. Performing the necessary operations, there result the following equations: Mr. LaChicotte.

$$182\,930\,X_5 + 165\,309\,X_4 = 5\,400\,509;$$
$$25\,668\,X_5 + 26\,666\,X_4 = 797\,015.$$

Whence, $X_5 = 19.29$, and $X_4 = 11.3$;

also, $X_3 = 34.363 - (X_5 + X_4) = 3.77$.

The resulting moments on the several members of the bent, as well as the positions of the points of contraflexure, are shown in Fig. 13.

These results do not differ materially from those of the author, but the resulting moments on the columns are uniformly greater at the top and less at the base, owing to the less degree of fixity at the base than was assumed by the author. It is noticeable also that the point of contraflexure in the upper column of a two-story bent is considerably lower (nearly 6 ft.) than in a one-story bent.

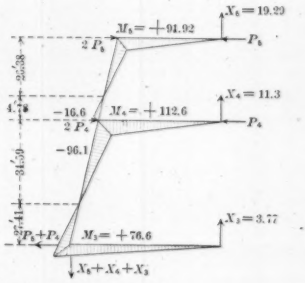


FIG. 13.

The position of the point of contraflexure depends upon the relative fixity of the ends of the individual member.

Owing to the symmetry of the bent, the ends of the struts are equally fixed, and the points of contraflexure are at their middles. The ends of the columns, however, are not equally fixed, as a rule, and the points of contraflexure will be found nearer the smaller moment.

If the struts at the bases of all the smaller towers are those used at the corresponding panel points of the large tower, counting from the top, no very serious errors will be found in the stresses of the smaller towers; but if, as in the case of the large tower, the bottom struts have been made slender as compared with those above, the resulting stresses, when these struts are included in the system, will be found to differ substantially from those given by the author.

The erroneous assumption of fixity at the base of the columns, and the failure to extend the calculations to the bottom struts, have led to serious errors in the large tower.

Here, the bottom strut has a moment of inertia of 2 216 quad. ins., while the half length is 51.02 ft. The moment at the base of the column is:

$$2\,704.892 - 51.02\,(X_5 + X_4 + X_3 + X_2 + X_1 + X_0)$$

whence, $$X_0 = \frac{2\,704.892 - 51.02\,(X_5 + X_4 + X_3 + X_2 + X_1)}{51.02}$$

Mr. LaChicotte. which must be taken negatively, as it produces a left-hand or negative moment; hence,

$$-X_0 = -53.016307 + (X_5 + X_4 + X_3 + X_2 + X_1) \ldots\ldots\ldots\ldots (a)$$

$$M = -X_0 x; \quad \frac{\delta M}{\delta X_0} = -x; \quad \int_{x=0}^{x=51.02} M \frac{\delta M}{\delta X_0} \delta x = 44\,269\ X_0.$$

Substituting the value of X_0 from (a) there results:

$$\int_{x=0}^{x=51.02} M \frac{\delta M}{\delta X_0} \delta x = -2\,346\,979 + 44\,269\ (X_5 + X_4 + X_3 + X_2 + X_1)$$

The value of this integral, when divided by the moment of inertia of the lower strut, must be added to each equation of condition. Multiplying each equation by 107 500, the value of the largest moment of inertia, and grouping the values of X_5, X_4, X_3, etc., there result the following five equations, containing five unknown quantities:

$$4\,530\,396\ X_5 + 4\,512\,770\ X_4 + 4\,365\,420\ X_3 + 3\,957\,800\ X_2 + 3\,361\,799\ X_1 = 222\,736\,629.$$
$$4\,512\,770\ X_5 + 4\,519\,221\ X_4 + 4\,365\,420\ X_3 + 3\,957\,800\ X_2 + 3\,261\,799\ X_1 = 222\,469\,029.$$
$$4\,365\,420\ X_5 + 4\,365\,420\ X_4 + 4\,383\,375\ X_3 + 3\,957\,800\ X_2 + 3\,261\,799\ X_1 = 217\,953\,209.$$
$$3\,957\,800\ X_5 + 3\,957\,800\ X_4 + 3\,957\,800\ X_3 + 3\,989\,471\ X_2 + 3\,261\,799\ X_1 = 202\,011\,089.$$
$$3\,251\,799\ X_5 + 3\,261\,799\ X_4 + 3\,261\,799\ X_3 + 3\,261\,799\ X_2 + 3\,318\,056\ X_1 = 170\,430\,318.$$

The values of the unknown quantities resulting are, $X_5 = 19.454$, $X_4 = 11.673$, $X_3 = 8.138$, $X_2 = 6.494$, $X_1 = 6.528$.

The value of X_0 is found from Equation (a), and is $X_0 = 0.729$.

The resulting moment, and positions of the points of contraflexure, are shown in Fig. 14, the notation of the author having been followed, in order to facilitate comparison with the results given by him. The effect of the author's assumption is strikingly illustrated in the case of the columns of the first story, where the moments at the top are increased about 70%, and in Strut No. 1 where there is an increase in the moment of about 32 per cent.

It is to be noticed also that the point of contraflexure in the column of the first story is to be found about 20 ft. below the middle point, while the moments given by the author show it about 4 ft. above the middle. This illustrates the danger of arbitrary assumptions as a basis of computations, and points clearly to the duty of using simple determinate forms unless conditions are such as to require otherwise.

The increased moments in the first story will run the unit stresses considerably above the limits fixed by the author, and would seem to necessitate the use of diagonal rods, or lower struts of greatly increased moments of inertia, to correct the evil.

It will probably be noticed that while the author has expressed the moments in foot-tons, the moments of inertia of the several members have been expressed in quad. inches. This is a very fruitful source of

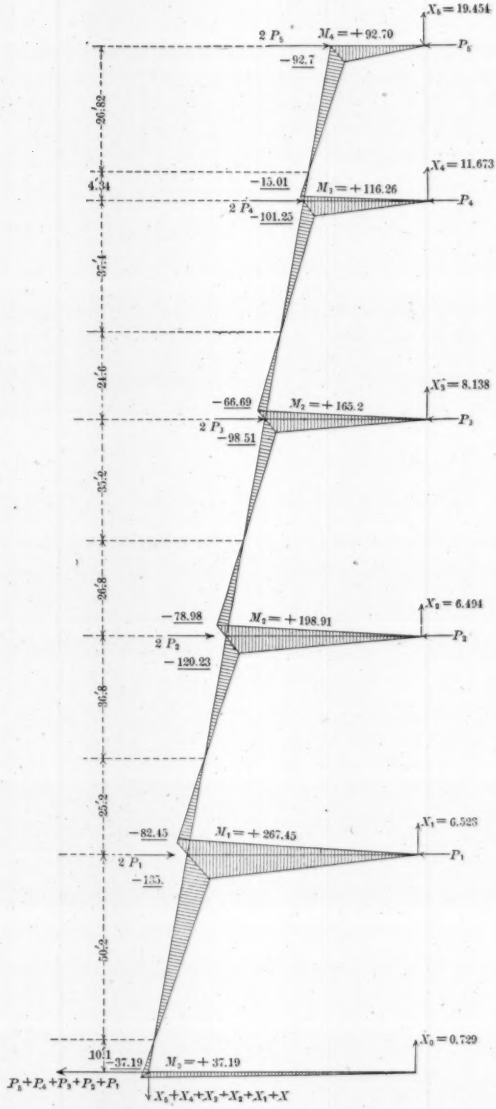

$X_5 = 19.454$
$2\,P_5$
$M_4 = +92.70$
P_5
-92.7
36.82
$X_4 = 11.673$
4.34
-15.01
$M_3 = +116.26$
P_4
$2\,P_4$
-101.25
37.4
24.6
$X_3 = 8.138$
-66.69
$M_2 = +165.2$
P_3
$2\,P_3$
-98.51
35.2
26.8
$X_2 = 6.494$
-78.98
$M_2 = +198.91$
P_2
$2\,P_2$
-120.23
36.8
25.2
$X_1 = 6.523$
-82.45
$M_1 = +267.45$
P_1
$2\,P_1$
$-135.$
56.2
10.1
$X_0 = 0.729$
-37.19
$M_0 = +37.19$
$P_5+P_4+P_3+P_2+P_1$
$X_5+X_4+X_3+X_2+X_1+X$

FIG. 14.

Mr. LaChicotte. error in ordinary computations of deflections, but in the present case no error results because it is the relative and not the actual values of the several moments of inertia which affect the results.

The work of deformation in a truss or beam may be produced by a moment, direct stress, shear or torsion, in any combination, depending upon the manner of application of the external forces.

In ordinary bridges torsion is avoided or reduced to a minimum; so that its effect may generally be neglected. In trusses the shear is usually replaced by direct or axial stresses, and in beams the work of deformation due to shear is very small as compared with that due to direct stress or bending; so that the effect of shear, as such, may usually be neglected without sensible error.

Under ordinary conditions, the only forces that need be considered as producing deformation are those of bending and direct stress; so that the work of deformation, which must be a minimum, may generally be expressed by the formula

$$W = \int \frac{M^2\, dx}{2\, E\, I} + \int \frac{P^2\, dx}{2\, E\, A} = \text{Min.}$$

Where M = the moment of the external forces;
E = the modulus of elasticity;
I = the moment of inertia of the section;
P = the axial or direct stress acting upon the section the area of which is A;
and dx = the element of length.

It is a matter of common knowledge that a force acting on a bridge member of ordinary proportions will distort it much more by bending than by either direct tension or compression, and it requires more material to resist the action of bending than that of direct stress. This is but another way of saying that the work of deformation of an axial force is very small as compared with that of a moment, and the author properly recognized this fact in dropping from the foregoing equation the expression for the work of the direct stresses, reducing the equation to the form

$$W = \int \frac{M^2\, dx}{2\, E\, I} = \text{Min.}$$

But in so doing he must have realized fully that by omitting the diagonal bracing of the towers it became necessary to introduce into the tower legs to resist the bending, a sectional area many times that of the discarded diagonals. Nearly one-half the sectional area of the tower legs is required to resist bending, while a small part of this additional area would suffice for an effective system of diagonal rods.

The speaker fully realizes that the lighter structure is not necessarily the cheaper, as there are many other considerations besides that of weight of material. Facility of erection is important, but the data

given by the author do not point to undue economy in this respect, even after allowing for the removal of the old structure. Uniformity of details has an important bearing upon the cost, both of manufacture and erection; but uniformity is very often secured by too great a sacrifice of material, and the ultimate cost of the structure thereby increased rather than reduced. If the contractors furnished the work in this instance at a pound rate, they are to be congratulated for devising a type of structure involving a minimum shop cost and a maximum tonnage. Mr. LaChicotte.

GUSTAV LINDENTHAL, M. Am. Soc. C. E. (by letter).—The first Kinzua Railroad Viaduct (built of wrought iron) attracted world-wide attention from engineers at the time of its construction (1880–81), because of its great height and the rapidity of its erection. Its articulated design of short spans—expansion joints at every tower, and the towers made up of braced bents, which were subject to large bending strains in the direction of the track—was not favorably thought of at that time by some engineers, who believed that the rigidity and durability of the viaduct were impaired thereby. Experience has proved the prediction to be correct. It is well known that, almost from the beginning, trains over that structure were obliged to slow down to about 5 miles per hour. Even then the vibrations, although not dangerous, caused unpleasant sensations. Mr. Lindenthal.

The old structure was typical of the American form, as distinguished from the European form, of metallic viaducts. In the latter the aim was always to obtain great rigidity, so that trains could pass over them without diminution of speed. Compact towers, continuous girders and longer spans were the leading features. Long-span arches also occur as parts of such viaducts over deep ravines, as in the Garabit Viaduct, in France, and the Münster Viaduct, in Germany.

It is a singular fact that in American bridge specifications, up to this date, as far as known to the writer, there are no provisions insisting upon rigidity of structure. If obtained, it is accidental, or the result of extra provisional care by the designing engineer, which he is under no obligation to give. The old Kinzua Viaduct complied in all respects with the old Erie specifications, which were then considered among the best. Nevertheless, it was a swaying structure, unfit for trains at ordinary speeds, and inferior to similar structures in Europe, built ten and twenty years earlier.

Among the earliest was the well-known double-track viaduct at Freiburg, in Switzerland (built in 1862), about 225 ft. high and 1 100 ft. long. The Iglava Viaduct in Austria, the Ossingen Viaduct in Switzerland, and others of considerable height and length, followed the Freiburg example. Faulty as some of their details are, in the light of present engineering knowledge, the essential fact remains that even at that early day it was found practicable to build high iron viaducts

Mr. Lindenthal. of sufficient rigidity to carry trains without diminution of speed. The weight of locomotives and cars on these viaducts since then has also increased, but they have stood wear and tear very well, and, so far as known, will, with few repairs, be serviceable yet for a number of years.

In the American construction, greater importance was attached to the temporary advantage of rapid erection, without falseworks, and with the fewest men, far away from the bridge shops. It led to a type of thin-legged and short-span trestle structures of flimsy appearance, inferior in rigidity to the European type. The trestle type may answer well enough for low heights, but is not suitable for greater heights, as proved by the old Kinzua Viaduct, and by a few other American railroad viaducts of less height which could be named.

The weight of locomotives and cars on the Erie Railroad has nearly doubled in the last twenty years, and the viaduct might have had to be rebuilt in any case; but, in doing so, the time had arrived also for a rigid structure over which trains could be run at full speed. This should have been the leading thought in the new design. It appears from this paper, and from the character of the plans, that such was not the case.

Because rolling stock has doubled in weight in the last twenty years, it does not follow that it will double again in the future. The limit will soon be reached (by reason of the crushing strength of hard steel in rails and wheel tires). An increase of 50% in the weight of the present heaviest rolling stock is improbable. But if it should occur, that margin is amply covered by the working stresses of the present Erie specifications. The new structure, under these conditions, should have been designed to be a permanent one, to last centuries.

For an intelligent judgment of the plans, Mr. Grimm does not furnish sufficient data. There is, for instance, no elevation or diagram of the viaduct, showing the arrangement and location of the expansion joints. The description of them on page 24 is not intelligible without such elevation. No weights of the new structure are given, from which the economy of design could be judged, in view of the author's allegation that the choice of the peculiar system of wind bracing in the towers was prompted by economy, when compared with other systems of bracing. Neither are data given as to the behavior of the structure under trains.

Some remarkable assumptions have been made. For instance, below the diagram (Fig. 8), showing the stresses in the tower bracing from braking, is a note, as follows:

"The computation of the stresses from the braking of trains assumes the point of contraflexure at half the height of the towers."

Mr. Lindenthal.

From which it appears that the towers are assumed to have at the top bending moments of the same value as those at the bottom, where the tower legs are anchored. How these are possible with the shallow plate-girders of the superstructure (of different heights and discontinuous chords) requires an explanation.

The horizontal force from a braking train per tower is not stated on the diagram, but may be judged, from the strains given for the stiff web members, to be about 50 000 lbs. for one side, or 50 tons per tower. For prudential reasons, the entire height of the tower, and not half its height, must be taken as the leverage for that force. The placing of an expansion joint in the superstructure at every third or fifth tower has not the slightest influence on that condition. The bending moments at the base will, therefore, be double those assumed; and there can be no bending moments at the top of the towers worth considering. Whether the columns and anchorages at the foot of the high towers are sufficiently strong to resist those larger bending moments cannot be judged from the meager data in the paper.

There are the further questions: How much do the towers bend or deflect under the horizontal braking force applied at the top? And does not the bending of the towers subject the riveted connections (of the track girders) at their tops to large bending strains, for which these connections are not adapted? Would not the high towers bend so much under a suddenly braked train as to cause the girders to slip off the roller bearings at the expansion joints, and cause the viaduct to collapse? Is not the only safety against such an occurrence in the continuity of the track structure?

These important questions can be accurately answered only with the aid of deflection diagrams of the towers, which are not given.

The most characteristic feature of the towers, however, is in the peculiar lateral bracing, called ordinarily the wind bracing, because (except on curves) it is usually dimensioned only for wind pressure, assumed to act quiescently. But the fact cannot be overlooked that this bracing is also exposed to the lateral impact and vibrations generated by the moving trains. These strains are dynamic in their nature, and therefore more dangerous on that account. The impact and vibration increase as the square of the train speed. Every trackman knows from experience that on a shaky structure trains must run slowly, although he may not know the theoretical reasons for that necessity.

Most bridge specifications make allowance for vertical impact by the addition of arbitrary percentages to the train-loads. A similar allowance should be made for horizontal impact, for the lateral vibration of fast moving trains, and for the violently alternating strains on the bracing. In ordinary bridges, these strains may safely enough be assumed to be included in the wind strains; but in high viaducts, or long spans, such an assumption is not warranted by observed facts.

Mr. Lindenthal. In the new Kinzua Viaduct, the lateral bracing is without diagonals in the towers. Stiff gusset connections with the horizontal braces take their place, on the plea that they are more economical. This may well be doubted. Does that arrangement produce a more rigid construction laterally? It is obvious that a deflection diagram of the tower bents at right angles to the track would furnish the emphatic answer "No."

Higher mathematics, as applied to the principle of least work, was probably never used for a poorer purpose than in the calculation of this tower bracing, needlessly made to be statically indeterminate, and which is inferior to the statically determinate braeing in the old towers, unsubstantial as the latter had been. The whole procedure shows a misconception of the fundamental conditions of good designing, and theoretical trifling with a serious problem, involving the safety of the structure.

Its details are also open to question. For instance, the large shearing strains in the tower columns from the bending moments at the gusset connections are taken up by the lacing angles. The single shear rivets of these angles, therefore, have to resist strains additional to those of ordinary lacing. The alternating strains from the horizontal impact and vibration expose the lacing angles to a sawing action. There is great likelihood here of the rivets loosening and endangering the stability of the towers. By turning the columns one-quarter around, so that the solid web plates would have taken the shearing strains, that danger, at least from loose rivets, would have been avoided.

But, after all, the defects in the bracing may be readily cured by the insertion of additional triangular or diagonal braces. The necessity for them in the higher towers will arise sooner or later.

If the new viaduct should seem at present to be stiffer than the old one, it can be so only on the low bents and under slow trains, and by reason of the greater weight and bulk of the new structure.

A rigid viaduct does not necessarily mean a more expensive one. Expansion joints should be the fewest in number. The structure is only about 2 000 ft. long, and dilatation joints placed at each end of the viaduct would have been sufficient to take care of temperature changes. The towers should not be subjected to the heavy strains from braking trains; they are not adapted for them. Resistance to braking trains can be furnished without them. Looseness of structure, lengthwise with the track, was a potent cause of the shakiness of the old structure.

If a competition of designs and bids had been invited, on the condition that the new structure must be suitable for fast trains, and that the sudden braking, or the starting up, of a double-header train must in no wise endanger its safety, then the proof would most likely have

been furnished, in a variety of designs, of its being readily attainable, and at no greater cost (if as great) than the structure described in this paper. Mr. Lindenthal.

F. H. Cilley, Esq. (by letter).—This paper raises many most interesting questions, both theoretical and practical. Mr. Cilley.

For the calculations, Mr. Grimm gives preference to an application of what is known as "the principle of least work," a principle so little known and less understood, even by many who use it, that a brief examination and exposition of it here may be of interest.

As commonly stated, this principle declares that "the actual stresses in a structure are those which make its work of deformation a minimum." But, set forth in these terms, the principle is in several ways most misleading when not actually incorrect. It suggests that it is based on the popularly accepted law that all natural processes tend to pursue the course most economical of energy. Yet, as this latter is not a true law, the principle, in so far as it is correct, cannot be simply a special application of it.

If one sought to interpret the law, ignorant of the rules for its application, one's impression would probably be that it asserted that the actual stresses (which are, of course, in equilibrium) represented less stored energy than any other set of stresses (not balanced, and therefore momentary), that is, that it asserted that the energy of the frame in the state of equilibrium under given forces was less than in any non-equilibrated state under these forces—a law of dynamics.

But, actually, it in no way states conditions of equilibrium, and is not a law of dynamics. What it really asserts is that the set of stresses in equilibrium under the given applied forces, which would wholly disappear on the removal of those forces (that is, which would occur under the applied forces if there were no initial stress), is that one among all possible sets of stresses in equilibrium under the given applied forces (for there are numberless such sets for all indeterminate structures), for which the potential energy of the structure is least. And the reason for this fact is of the simplest. For structures obeying Hooke's law, the distortion accompanying the application of any given set of forces is independent of the existence of initial stress, and, therefore, not only the change in stress, but the work done to the structure by the applied forces, and, therefore, the change in the potential energy of the structure, are independent of the existence of initial stress. Thus, the total potential energy of a structure in equilibrium under given applied forces is its initial potential energy (due to its initial stresses) plus the (invariable) change of potential energy due to the application of the given forces. The total potential energy evidently is least when the initial potential energy is zero, that is, when there is no initial stress, and thence the law as last

Mr. Cilley. stated. It is not a law of equilibrium; it is not a special case of a great universal law of Nature, but is simply a limited law of the theory of elasticity, dependent for its truth, even in the second form given herein, on whether Hooke's law happens to hold, or does not happen to hold, for the materials and manner of distortion of the structure under consideration. In its first form it is not necessarily correct, even where Hook's law does hold, for the stresses it determines are, in general, simply the changes in stress due to the given applied forces, which are only the actual stresses when there chances to be no initial or self-balancing stresses.

Now, as to its application: First, the potential energy of the structure under the given applied forces must be expressed in terms of any variable set of stresses in equilibrium with them, and including in their variation the set of stresses sought. This is done by noting how many more stresses have to be determined than there are conditions provided by statics. As many of the stresses as have thus been found to be in excess are then designated by letters as unknowns. They evidently may be independently given any arbitrary values whatever, and the conditions provided by statics still be satisfied. They are, therefore, independent variables. The conditions of statics now permit all the stresses in the structure to be expressed in terms of the applied forces and these unknown stresses. Thence it is very easy to express the corresponding potential energy of the structure. Now, for the solution sought, the set of stresses for which there is no initial stress, or the change in stress under the applied forces—the set of values of the unknowns—the independent variables—must be found, which will make the expression for the potential energy of the structure a minimum. This is purely a question of mathematics. As is known, the differential coefficient of the expression for the potential energy of the structure, with respect to each of the independent variables, must be zero. This gives as many equations as there are independent variables. Stress being proportional to strain (Hooke's law), the expression for the potential energy of the structure is quadratic in terms of the variables. These partial differential coefficients are therefore linear. The values of the independent variables, consequently, are determined simply by solution of linear equations. Such is, in outline, the complete explanation of "the principle of least work" and its application.

What are its merits and demerits as a working method? it may be asked.

It may be used almost mechanically, and therefore saves thought, but at the expense of understanding. Like most integral theorems, it is a sort of blanket which hides the details and prevents the perception of the significance and purpose of the procedure. On this account, the writer is inclined to prefer the use of a more objective

method, in which the structure is conceived as being so cut up or otherwise altered as to make it statically determined, place forces and moments at the various cut sections or otherwise altered points to replace the restraints removed, then, preferably, by aid of the method of virtual work (also known as the method of virtual velocities or displacements), determine the corresponding displacements, the equation of which gives the conditions which must exist in the original uncut and unaltered structure. In the case of no initial stress this results in precisely the same linear equations as before, * but here the meaning of each is clear; one is in close touch with the physical problem at all times; errors are easily discovered and corrected; the consequences of changes are readily perceived, and one gains a really intimate insight into the workings of the structure which is the object of the investigations. In particular, cases involving consideration of changes in temperature, yielding supports, errors in workmanship, and initial stresses generally, are dealt with clearly and readily. Mr. Cilley.

Turning from general considerations to the special problem offered by the design of the Kinzua Viaduct, consider the plane bent of five stories. By cutting through each of the five headers, say at the middle, separating the two halves and putting at each cut section the forces which had actually acted on it before the cutting, each of the halves would become statically determinate, and would offer no further difficulties. But these forces acting on the cut sections are precisely those which are not known, but have to be determined. At each section there are three of these, a shear, an axial stress and a moment, or fifteen in all. Indicating each of these by a letter, there would be no difficulty in writing out the expression for the potential energy of the whole bent in terms of the applied forces and these fifteen unknowns. Partially differentiating with respect to each of these unknowns, and placing each result equal to zero, fifteen linear equations between the fifteen unknowns are obtained, the solution of which yields their values.

But in Mr. Grimm's treatment not fifteen but only five unknowns appear, and only five equations are found. This is in consequence of

* The potential energy of a structure under load may be expressed by $\frac{1}{2} \sum S \Delta$ — one-half the sum of all the products of the stresses into the corresponding strains—if the material obeys Hooke's law. The strains Δ are proportional to the stresses S, $-\Delta = KS$ say—so that the potential energy of the structure becomes $\frac{1}{2} \sum KS^2$. Now, if the structure be statically indeterminate, the S are linear functions of the superabundant stresses X. The values of these, if there are to be no initial stresses, it has been seen, must satisfy the equations obtained by partially differentiating the energy equation, $\frac{1}{2} \sum KS^2$, with respect to each of them, bearing in mind that they are independent variables, and equating each result to zero. That is, the X are determined by solution of the simultaneous linear equations: $\frac{\delta (\frac{1}{2} \sum KS^2)}{\delta X} = \sum KS \frac{\delta S}{\delta X} = 0$.

But the expressions $\frac{\delta S}{\delta X}$ are the stresses S^1 which would result, if the value unity were given to the X in question, all other X being zero, and the equations become: $\sum KSS^1 = 0 = \sum \Delta S^1$; a form familiar to all who use the method of virtual work, which at once expresses the fact that the difference in displacement in the direction of X of the two sides of the section on which the stress X works, is zero.

Mr. Cilley. his neglect of certain factors. If the complete equations were written out, denoting by Y and Z the axial stress and central moment, respectively, of each header, it would be found that, owing to the symmetry of the structure and the character of the loading, the equations $\frac{\delta F}{\delta X} = 0$ do not contain either the Y or the Z, while the equations $\frac{\delta F}{\delta Y} = 0$ and $\frac{\delta F}{\delta Z} = 0$ do not contain the X; F representing the expression for the potential energy of the bent under the given loading. Mr. Grimm considers only the first set of equations. The second and third sets determine values of Y and Z largely dependent on the compressibility of the headers. Assuming the headers incompressible (although flexible) the Y and Z become zero, and this, at least, is involved in Mr. Grimm's solution. He further treats the columns as incompressible and inextensible, although flexible. Mr. Grimm's statements in this connection on page 29 are very misleading.

Turning to Mr. Grimm's actual work, it may be noted that he has integrated for the vertical distance between headers, instead of for the true oblique lengths of the columns. These and other errors in his work, made apparent by a calculation of the deflections from his results, which wholly failed to check, made it futile to attempt any thorough discussion of Mr. Grimm's design from his own data, and the computation of correct data is too laborious.

On two points, however, it is important to arrive at some sort of a conclusion: As to the economic necessity of the design used, and as to its stiffness.

As to the first of these, the writer finds that by Mr. Grimm's data the unit stress in the headers rises to some 12 000 lbs. Taking this as a basis, and the column formula as published, the writer computed a stiff system of diagonal bracing at about 45° with the vertical, and found it at least as light as the headers. So that, even without considering the removal of the secondary stresses in the columns (some 5 000 lbs. per square inch) resulting from the use of the stiff diagonal system, it is at least as economical as that actually used, thus showing that Mr. Grimm was mistaken in supposing his design one required for economy.

As to the second, it apparently does not require argument to demonstrate that a completely braced frame must have smaller deflections, for the same unit stresses, than a frame without diagonals, resisting only through the bending of relatively slender members. Mr. Grimm, at least, can hardly take exception to this conclusion in this case, for in his work he has gone on the basis that the distortions due to bending are so great that those due to axial stress are by comparison wholly negligible. Either this is correct or Mr. Grimm's computations are worthless.

C. R. GRIMM, M. Am. Soc. C. E. (by letter).—In presenting a paper on the Kinzua Viaduct the writer had hoped that its discussion would produce some points of value, either of a practical or theoretical nature, but in this he has been disappointed. Mr. Grimm.

If the design had been subjected to a critical and scientific examination, facts would have come to light instead of opinions.

While an answer to the minor objections of the critics can be found in the paper, and need not be touched upon again, the writer will confine himself to answering more fully the objections on which the discussors lay the greatest stress, as this is, in any case, the object to be attained.

Answers to objections of the same nature, although raised by several discussors, will, of course, be given but once.

Some mathematical notes in regard to the analysis have been added in order to meet the wishes of some of the discussors and give a proof of certain statements made by the writer.

Although Mr. Buck failed to do justice to the practical part of the design, the writer gives him credit for his persistent effort, with able outside assistance, to follow the analysis, but cannot agree with him that sufficient data for a discussion have not been given.

As previously stated, the writer assumed the reader's familiarity with the principle of least work, and engineers acquainted with modern analytical methods will find the data sufficient as a basis for a thorough discussion. It has been stated in the paper that wind stresses alone would be examined, and, of course, by some analytical method. The assumption of the points of contraflexure in the graphical method is arbitrary, while in the analytical method this assumption is not and must not be arbitrary. As a matter of fact, no assumption whatever has been made in the analysis that cannot be proved to be justifiable; but this point will be taken up at the proper place.

In reference to the closing paragraph of the paper, it should be stated that by a group of members is meant either the columns or the cross-struts, and, although the effect on the resulting moments of a change in the moments of inertia of a group of members between rather wide limits is quite small, the effect on the stiffness of the frame may or may not be small. This depends upon how the increase in the moments of inertia is effected; if, for instance, by increasing the depths of the struts or of the columns, then, of course, the bent will gain in stiffness.

According to the figures on the stress sheet, there is no up-lift on the windward column, as erroneously stated by Mr. Buck; but the writer agrees with him in the rejection of the old anchor bolts. Strong bolts should have been used, with superior details, and, therefore, much greater efficiency.

Mr. Grimm. Mr. La Chicotte laments over both the length of the analysis, as the principal feature of the paper, and the shortness of the description of the viaduct, but, to judge from his vigorous complaints that so much has been left unexplained in the analysis, one is led to the belief that the length and breadth of the analytical method is far behind the requirements of Mr. La Chicotte, and it is significant indeed that his whole discussion centers in the bottom transverse strut. Mr. La Chicotte, in conjunction with Mr. Moisseiff and Mr. Buck, objects to the writer's assumption of the fixity of the bents.

While perfectly aware of the fact that no connections whatever are fixed, in the scientific sense of the word, the writer, nevertheless, is unshaken in his view that, at least in the case under consideration, this assumption answered all practical purposes.

If the discussion was meant to be logical, why, then, was not the writer questioned with regard to the different degrees of fixity of the bents? Why was he not questioned about the influence on the moments of the varying heights of the bottom stories of the towers; and, consequently, why was he not expected to undertake forty sets of computations instead of the five actually made? And why was no inquiry made regarding the assumption of concentrated wind pressures? Drawing the final conclusions, which Mr. La Chicotte from some reason ignores, we arrive at the following results.

Assuming his figures to be correct, then the bending stress in the bottom column of the highest bent, according to his own data—and no other members need be considered—is increased 1 600 lbs. per square inch, which means that the computed wind stress, by Mr. La Chicotte, could be increased 360% before the combined stress from the different loadings would reach the elastic limit, not considering the reducing effect of an 8-ft. strut on the moments of the column.

While an inquiry into the assumption of fixity is quite proper, it is nevertheless true that Mr. La Chicotte has made a mountain out of a molehill, and the work spent on the bottom strut should have been directed toward some more profitable point.

Mr. La Chicotte claims that he has worked out the problem for himself. The only thing he has done consisted in trifling additions (on account of the transverse bottom struts) to the writer's original condition equations, and solving them over again. The insufficient and partly erroneous general theoretical remarks, together with his statements about the points of contraflexure, which are bound to lead to erroneous views, fully explain that he did not digest the problem.

A solution of the problem on independent lines would have been very welcome to the writer for the sake of comparing results, but no such solution has been forthcoming during this discussion.

The writer agrees with Mr. Lindenthal when he questions the advisability of building very high viaducts. A viaduct of 300 ft. in

height and with spans of some 40 and 60 ft. has already passed the limits of economy and desirability, no matter how it is designed, and, in the opinion of the writer, in the present case from a purely economical point of view, its place should have been taken by an arch. A spandrel-shaped arch, with its horizontal top chord, could hardly be considered seriously, as it is neither fit for a very long span nor for a high rise. Of all the different types of arches, the writer, in all probability, would have selected as the most suitable the crescent form, the chords of which intersect at the springing. This form is the most economical among all the two-hinged statically indeterminate arches, and is excellently adapted for a long span and a high rise. With moderate temperature stresses it combines great uniformity in the chord stresses, so that each chord could be built of uniform section from end to end and without any great waste of material. The approaches could be viaducts, the erection of the arch be effected cantilever-fashion, and the trusses be given an inclined position, whereby any amount of lateral stability is obtained. Mr. Grimm.

Mr. Lindenthal's idea, to have the Kinzua Viaduct designed to last for centuries, cannot be taken seriously, for very obvious reasons.

The doubling of the traction stresses, as Mr. Lindenthal thinks, is not warranted by the facts. To a given traction stress corresponds a longitudinal deflection of the top of the tower, and, while the play of the intermediate spans is sufficient for the changes of temperature, it is entirely inadequate to double the traction stresses.

One of his statements in connection with this subject reads: "There can be no bending moment at the top of the towers worth considering;" and, a few lines below, there is the following question:

"And does not the bending of the towers subject the riveted connections (of the track girders) at their tops to large bending strains, for which these connections are not adapted?"

The writer believes that Mr. Lindenthal will find it hard, if not impossible, to give a satisfactory explanation for this bare contradiction.

A reversal of stresses in some parts of the towers occurs only under extreme conditions and at rare intervals, so that Mr. Lindenthal's apprehensions are more imaginative than real. The change in length of the track stringers, due to a change of temperature of 75° Fahr., is about 10 ins., and, since the structure is only about 2 000 ft. long, Mr. Lindenthal thinks that one dilatation joint at each end is sufficient to take care of the longitudinal motion, but as he fails to explain in what manner these expansion joints are supposed to annul the effect of a longitudinal force of about 800 tons on the viaduct, the writer cannot discuss his suggestion. His cloudy views of, and prejudice against, the general design, have prompted him to a criticism which is not justified by the facts involved in this case.

Mr. Grimm. With regard to the omission of diagonals in the transverse faces of the towers, he asks:

"Does that arrangement produce a more rigid construction laterally? It is obvious that a deflection diagram of the tower bents at right angles to the track would furnish the emphatic answer 'No.'"

A mere dictum is not equivalent to a proof. No engineer will earnestly dispute the fact that he only is entitled to an extreme judgment who combines his practical experience with the most searching analytical examination of a structure, by which means its true nature is disclosed. Mr. Lindenthal has not even given the least demonstration of any sort, not to mention a rigid mathematical discussion, to prove the truth of his assertion.

In view of these facts the writer rejects the above quoted statement as also other bold but groundless statements, as worthless.

Mr. Cilley complains about the nature of the analysis used. To characterize his standpoint it will be best to quote his own statements from different sources. In the discussion of his paper on Indeterminate Frameworks,* he expresses himself as follows:

"The use of methods of calculation based on the principle of virtual 'Velocities' or 'Displacements' or 'Work,' as it is variously termed, or which result from the principle of least work, which is simply an integral form of the preceding, is the simplest as well as the most correct means of determining changes in stress and strain in indeterminate structures. Where such structures exist, and, therefore, must be examined, the writer most strongly approves the use of these methods. And in designing, if engineers will use indeterminate forms, these methods are still to be preferred."

Apparently, his mind has undergone a remarkable change in reference to the principle of least work, for he writes in the present discussion regarding it:

"What are its merits and demerits as a working method? it may be asked. It may be used almost mechanically, and therefore saves thought, but at the expense of understanding. Like most integral theorems, it is a sort of blanket which hides the details and prevents the perception of the significance and purpose of the procedure."

In respect to the diffusion of the principle of least work, Mr. Cilley is quoted from the first-named source, when he says of himself: "The writer's point is that while the methods just referred to are the best known," etc., but in the present discussion he speaks of the principle of least work as of "a principle so little known and less understood," etc.

It is to be regretted that he did not put on record some illustrations of "almost mechanical" applications of the principle of least work in the solution of engineering problems. Comments on his contradictions are superfluous.

* *Transactions*, Am. Soc. C. E., Vol. xliii, page 426.

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Mr. Cilley accuses the writer of having committed an error on account of the integrations for the vertical heights instead of for the lengths of the columns. The writer did integrate for the vertical heights for the sake of simplification, which involved no error at all in that sense underlying Mr. Cilley's remark. Let us assume a beam, loaded at its extreme end with a horizontal force P (see Fig. 15). The determination of the horizontal deflection δ in the direction of P requires an integration for the axis of the beam, but, if the angle α is not too large, we integrate for the vertical height for the sake of simplification, and while it is true that an error has been committed, for all practical purposes, this error can be neglected as being too small to be considered. In the examination of flat arches, for instance, we proceed in like manner and use the differential dx instead of the arc element ds. Similar reasons prompted the writer to perform the integrations as he did. The same misleading statement is repeated by Mr. Cilley in regard to the unknowns of the bents, when he says of the writer: "This is in consequence of his neglect of certain factors."

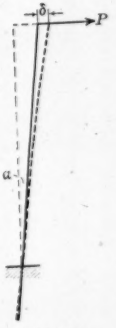

FIG. 15.

Mr. Cilley seems to be unconscious of the fact that the influence of axial and shearing forces on the final result is small compared with that of the bending forces, a fact so widely known that it is wasting time to dwell on this subject longer.

We will now turn our attention to the omission of diagonals in the transverse faces of the bents and to the deflections of the towers, treating the latter as free standing cantilevers, although, strictly speaking, they are not.

Believing that a comparative computation of the deflections would be welcome, the writer examined the deflections due to bending of five different bents, the same as are described in the paper, and also the deflection of an articulated bent, corresponding in its height, 155 ft. 2 ins., to a three-story bent as built, but having five stories instead of three, for obvious reasons (see Fig. 17).

The wind load is, of course, the same for all bents, and equal to that assumed in the paper.

Each half of a bent is statically determinate; and, the exterior forces being all known, the deflection can be determined.

The calculations of the deflections proceed from the "Theorem of the Derivative," which is expressed as follows: The displacement, in the direction of an external force, of its point of application, equals the partial derivative with respect to the force of the work of deformation of the frame.

Mathematically expressed, the deflection D_1 of the stiff bent sub-

Mr. Grimm.

jected to moments and D_2 of the articulated bent appear under the form

$$D_1 = \int \frac{M}{EI} + \frac{\delta M}{\delta P} \, dc \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and

$$D_2 = \Sigma \frac{S L}{E A} + \frac{\delta S}{\delta P} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

In Equation (2), S designates the stress in any bar; L the length of that bar; and A its sectional area. The other letters in Equations (1) and (2) have the same meaning as in the paper.

The stress S can be expressed:

$$S = \alpha_1 P_1 + \alpha_2 P_2 + \alpha_3 P_3 + \ldots . \alpha_m P_m + \ldots . + \alpha_n P_n;$$

and the partial differential coefficient, with respect to any of the external forces, as for instance P_m, would be:

$$\frac{\delta S}{\delta P_m} = \alpha_m,$$

α_m is the stress in any bar of the frame, if all the exterior forces P are supposed to be removed, excepting P_m, for which is substituted a force = 1.

The meaning of α_1, α_2, α_3, etc., is similar to that of α_m.

The term $\frac{S L}{E A}$ gives the alteration in the length of any bar in the frame, and if we substitute for this term the play in the pinholes, Equation (2) can be used to determine both the elastic and the non-elastic deflection. In Fig. 17 the stresses "α" are put in brackets.

The integrations or summations, as the case may be, must be extended over the whole frame.

Since the lengths are given in feet and the forces in tons for the stiff bent, the moments of inertia must be reduced to feet and the modulus of elasticity to tons and the square foot.

The deflections of the tops of the five bents, computed from the data given in the paper, are as follows:

1-story	bent,	31 ft.	2 ins.	high	on	centers,	$\frac{1}{16}$ in.
2	"	93 "	2 "	"	"	"	$\frac{13}{32}$ "
3	"	155 "	2 "	"	"	"	$\frac{9}{16}$ "
4	"	217 "	2 "	"	"	"	$1\frac{1}{32}$ ins.
5	"	279 "	2 "	"	"	"	$2\frac{1}{4}$ "

The total deflection, elastic and non-elastic, of the articulated bent amounts to $1\frac{5}{32}$ ins.; the play in the pinholes is supposed to be $\frac{1}{32}$ in., while that of a three-story riveted bent of triangular form is only $\frac{9}{16}$ in.

The assumed sections of the columns in the articulated bent are the same as those in the riveted one, but, if such a frame had been built, no such sections would have been used; on the contrary, the

Mr. Grimm.

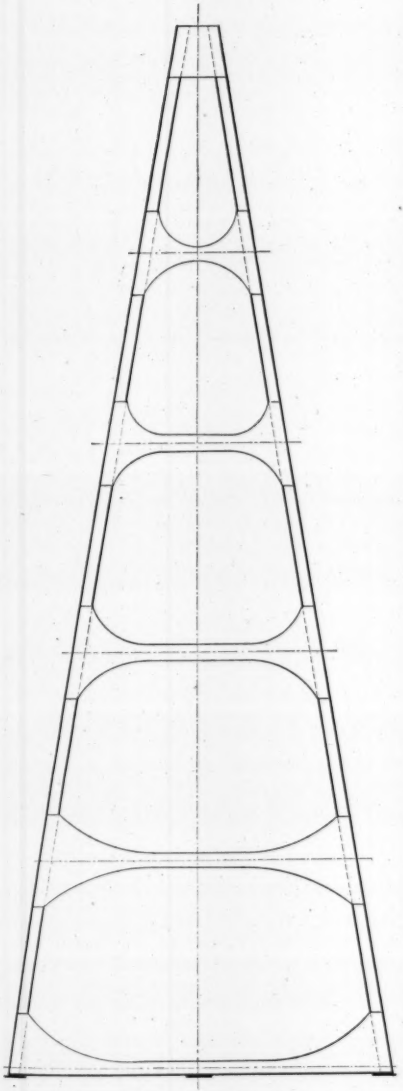

FIG. 16.

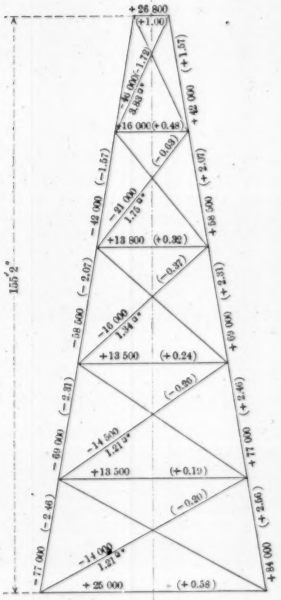

FIG. 17.

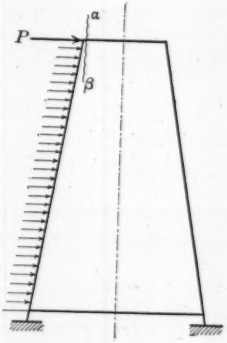

FIG. 18.

Mr. Grimm. column sections would have been much smaller, corresponding to a larger deflection than that of $1\frac{15}{32}$ ins.

The deflections compare very favorably indeed with those of any other form of bent, articulated or not, and this circumstance is due to its triangular form and deep and very rigid header.

Every frame has somewhere a critical point, so that if a force is applied at this point, it will show its least resistance to distortion.

For a bent of the usual form, this critical point is at the top, and, unfortunately for this bent, it is at the top where the principal wind forces, as well as all the dynamic forces, are applied.

This statement is not applicable in its full force to the present bent, because a riveted triangular frame shows its greatest resistance to distortion if forces are applied at the apex.

While it is apparently evident in itself for the critics in general, requiring no demonstration of any sort, that a triangular bent deflects very considerably more than an isosceles trapezoid, the mathematical examination shows this view to be erroneous. Articulated high bents are capable of large swaying motions, as it were, but a stiff and massive triangle, the simplest form of a bent, with a very rigid apex, the principal point of application of the forces, properly designed, with due regard to minimizing the bending stresses and the deflections, will perform legitimate vibrations.

The wind stresses in the transverse diagonals are comparatively small, and decidedly so toward the bottom of the towers. The questions may now be asked: Is it really such an undertaking, as some of the critics have imagined it to be, to replace these diagonals, with their small stresses, by massive fixed connections, at the same time changing the bent, a very important and very advantageous step, into a triangle? And is a frame of a few, but very substantial, members not more fit to stand the effect from the passage of trains, than a frame of very many but rather light members?

If the present bents would leave something to wish for, that, in the writer's opinion, would be no argument against their use, because every design is capable of improvement.

Although the workings of a frame can only be judged by the analysis, Mr. Morison nevertheless clearly recognized the importance of the principal features of this design, when he made the statement that it is possible to build such a structure perfectly secure; and, coming, as it does, from so distinguished an engineer, it commands respect.

It is a well-known fact that the requirements of economy call for the least number of substantial members in a structure, instead of for a multitude of slender pieces; and, in order to make a fair comparison, bents as well as towers should be compared with respect to weight, ease and quickness of manufacture and erection, etc.

It does not seem to need an argument to show that the sub-

stitution of two compression diagonals for an intermediate strut can only be made at an expense of metal—and in point of weight it would be still worse if two diagonals together with a strut were used—as the average combined length of the two diagonals in each story is about three and one-half times as great as the length of the strut. The weight of the web members in the transverse faces of high bents of ordinary construction is entirely out of proportion to the stresses in these members, and the weight of that mass of longitudinal secondary bracing used as a prop to prevent the transverse members from a collapse, is a very large item indeed in tall towers. In such a case, where is the economy? It must also be borne in mind that in the present case a considerable weight could have been saved if the towers had not been built all alike in their sections from their tops down. Mr. Grimm.

While stresses and sections in a bent, as usually built, are entirely out of proportion, there is in the triangular form, on the whole, a far better harmony. The intermediate struts in the Kinzua Viaduct have been built with parallel chords, for practical reasons, giving them a surplus of metal around their centers, where it was not necessary. These struts do not need to be straight. Fig. 16 shows the highest bent with curved cross-pieces, supposed to be box-shaped, struts and brackets all in one piece and fork-like at the ends, so that they can take the firmest grip on the columns.

In a design like this the metal in the struts can be distributed so as to meet in the best possible manner the requirements of the stresses. It is also quite feasible to design a true triangle by making the longitudinal axes of the columns to intersect on the horizontal top edge of the header, the latter to be cantilevered to take the stringers.

The question whether a triangle can be so designed as to carry a double-track railway, is best decided by an investigation of the stresses and details.

An increase in the stability by using a greater batter involves a considerable outlay of money for the usual form of bent, but not so for a portal-braced triangle.

If intermediate struts with curved ends were used, as shown in Fig. 16, the increase in weight of these struts would be effected by lengthening the central parts, where the struts are the shallowest, and, although the length of the columns is somewhat increased, it must not be forgotten that the bending moments of the frame decrease—as also the deflections—with an increase of batter; so that, on the whole, a greater stability can be obtained with the portal-braced triangle at a lesser cost.

There are two more points not mentioned by the discussors.

Although we hear a good deal about the statical determinateness of towers, as ordinarily built, they are in reality statically indeterminate, and even, quite often, highly so.

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Omitting the intermediate struts in either the longitudinal or transverse faces, and using in each story two crossed diagonals and a strut at top and bottom, we have a singly statically indeterminate frame, and if, besides the diagonals, intermediate struts are used, as is done very often, we produce a frame that is n fold statically indeterminate, if there are n stories.

The second factor to be dealt with in such a frame is the bending stresses due to the fixed joints. As a matter of fact, the constructor completely ignores the statical indeterminateness in his computations by the simple supposition that it is non-existent, assuming frictionless pins instead of fixed joints.

Although it is true that some constructors minimize the bending stresses by skilful designing, others, apparently, do not pay the slightest attention to them.

In a case like that under discussion, all stresses must be examined, and herein the writer sees an advantage.

The second point deserving to be mentioned is the appearance of a portal-braced triangle, which is far ahead of that of a very slim and ugly looking tower with its many pieces, impressing one more with the idea of a huge bird cage than anything else. In concluding this discussion it will be necessary to add some notes with respect to frames, the diagonals of which have been replaced with fixed joints.

The frames considered are supposed to be symmetrical in form as well as in distribution of material about a vertical axis; and the forces applied to them are assumed to act in the planes of these frames.

In the paper the writer stated that a one-story bent, fixed at the bottom, is a threefold statically indeterminate structure. Assuming for the present that the exact values of the three unknowns in the top story of a two-story bent were known and these unknowns as exterior causes applied to the bottom story, there would still be three more unknowns left to be determined, so that we have six unknowns in all for a two-story bent. And, in general, a bent of n stories has $3n$ unknowns. The causes which may influence a truss of this kind are as follows:

1.—Bending forces.
2.—Axial forces.
3.—Shearing forces.
4.—Uniform change of temperature.
5.—Non-uniform change of temperature.
6.—Displacements of the points of application of the reactions.
7.—Misfits of the bars composing the frame.

The condition equation, covering the foregoing seven causes, is:

$$R = \int \frac{M}{I\,E}\,\frac{\delta M}{\delta X}\,dx + \int \frac{F}{E\,A}\,\frac{\delta F}{\delta X}\,dx + C\int \frac{T}{E_t\,A}\,\frac{\delta T}{\delta X}\,dx + \int \varepsilon\, t_o\,\frac{\delta F}{\delta X}\,dx + \int \varepsilon\,\frac{t_1 - t_2}{h}\,\frac{\delta M}{\delta X}\,dx.$$

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In this equation:

M = a bending moment;

F = a direct stress or axial force;

T = a shearing force;

I = a moment of inertia;

A = an area;

ε = the coefficient for extension or contraction due to a change in temperature of 1°;

t_0, t_1 and t_2 = temperatures according to certain specified assumptions and laws;

C = a constant, dependent on the form of a section;

E = the modulus of elasticity, expressing relations between longitudinal strains and normal stresses in the same direction;

E_t = the transverse coefficient of elasticity, expressing relations between distortions and tangential stresses in the same direction.

For steel, $$E_t = \frac{3}{8} E \text{ to } \frac{2}{5} E.$$

R is the work of the reactions done under the supposition that any one of the unknowns, $X = 1$.

The first integral refers to bending moments, the second to axial forces, the third to shearing forces, the fourth to a uniform change in temperature, as also to misfits, the fifth to a non-uniform change of temperature.

It is customary in a case like that under consideration to reject any possible influences due to the causes under 4, 5, 6 and 7. Of the remaining causes, under 1, 2 and 3, the bending forces are by far the most important, as the influence of axial and shearing forces on the final results is too small to be considered. A few examples may suffice to illustrate the nature of the bents.

The two-story frame, Fig. 20, is acted upon by two concentrated loads, one at each panel point, 4 and 2, respectively, and, in order to avoid extensive calculations, simple dimensions have been selected, with the assumption of a uniform moment of inertia. We will consider bending alone.

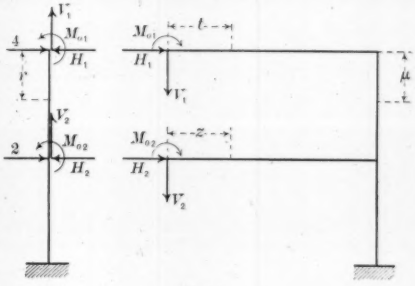

Fig. 19.

Passing a cut very close to the left column and applying at each cut and at each of the two fragments, but in opposite directions, a

Mr. Grimm. moment, a shearing force and an axial force (see Fig. 19), the equilibrium is re-established and the six condition equations can be written down at once and solved.

These six equations are:

$$\left.\begin{aligned} &\int \frac{M}{EI}\,\frac{\delta M}{\delta M_{01}}\,dx=0;\int \frac{M}{EI}\,\frac{\delta M}{\delta M_{02}}\,dx=0;\int \frac{M}{EI}\,\frac{\delta M}{\delta V_1}\,dx=0;\\ &\int \frac{M}{EI}\,\frac{\delta M}{\delta V_2}\,dx=0;\int \frac{M}{EI}\,\frac{\delta M}{\delta H_1}\,dx=0;\int \frac{M}{EI}\,\frac{\delta M}{\delta H_2}\,dx=0. \end{aligned}\right\}$$

Each of these six integrals must be applied to each bar of the frame, the results added together and put equal to zero, so that, in all, six new equations are obtained, from which the unknowns can be computed. To illustrate the foregoing statement, take as an example the partial differential coefficient of the work of deformation of the frame with respect to V_1. The force V_1 leaves Bar 2 entirely unaffected, produces a direct stress in Bars 3 and 4 and bending in Bars 1, 5 and 6.

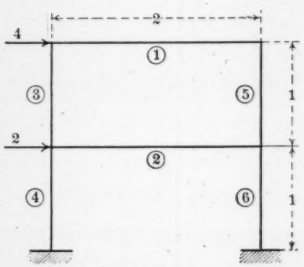

FIG. 20.

With this understanding, we have:

$$\int \frac{M}{EI}\,\frac{\delta M}{\delta V_1}\,dx=\int_0^2 \frac{M_1}{EI_1}\,\frac{\delta M_1}{\delta V_1}\,dt+\int_0^1 \frac{M_5}{EI_5}\,\frac{\delta M_5}{\delta V_1}\,du+$$

$$\int_1^2 \frac{M_6}{EI_6}\,\frac{\delta M_6}{\delta V_1}\,du=0.$$

The indices refer to the bars, and, under the above assumption,

$$I_1=I_5=I_6.$$

The resulting six equations, after performing the computation of the integrals, are:

$$\left.\begin{aligned} -18\,M_{0_1}-6\,M_{0_2}+32\,V_1+12\,V_2-12\,H_1-3\,H_2 &=0\\ -6\,M_{0_1}-12\,M_{0_2}+12\,V_1+20\,V_2-9\,H_1-3\,H_2 &=0\\ +12\,M_{0_1}+9\,M_{0_2}-12\,V_1-9\,V_2+16\,H_1+5\,H_2-37 &=0\\ +3\,M_{0_1}+3\,M_{0_2}-3\,V_1-3\,V_2+5\,H_1+2\,H_2-12 &=0\\ +6\,M_{0_1}+2\,M_{0_2}-6\,V_1-2\,V_2+4\,H_1+H_2-9 &=0\\ +2\,M_{0_1}+4\,M_{0_2}-2\,V_1-4\,V_2+3\,H_1+H_1-7 &=0 \end{aligned}\right\}$$

Mr. Grimm.

Solving for the unknowns we have:

$$V_1 = \frac{270}{228};\ H_1 = 2;\ M_{0_1} = \frac{270}{228};$$

$$V_2 = \frac{396}{228};\ H_2 = 1;\ M_{0_2} = \frac{396}{228}.$$

Fig. 21 shows one of the intermediate panel points in equilibrium under the influence of the exterior and interior forces. The small circles indicate the points of contraflexure, which, for the struts, it will be noticed, are at their middle. The compression in the struts equals one-half of the exterior concentrated loads.

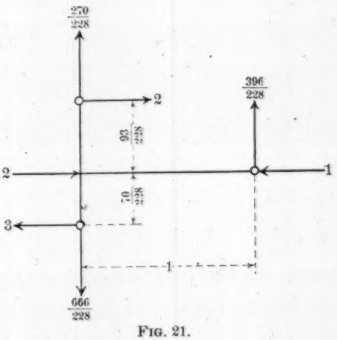

FIG. 21.

A variation in the moments of inertia between columns and struts, although of course influencing the bending moments, has no influence on the location of the points of contraflexure of the struts. But, if besides the bending forces, the axial forces had also been considered, the points of contraflexure of the struts would have been thrown somewhat out of their middle.

If the influence of bending forces alone is examined, a procedure perfectly legitimate and answering all practical purposes, and if it is known in advance that under this condition the points of contraflexure of the struts are at their middle and their compression equal to one-half of the panel point concentrations, a very considerable labor indeed can be saved by computing one unknown for each story instead of three, as the writer actually did. Passing a vertical cut through the center of the frame, Fig. 20, and applying V_1 and 2, as also V_2 and 1 to each half of the bent, but in opposite directions, disregarding the moments at the cut, as no moment can pass it, we arrive very quickly at the two equations:

$$\left.\begin{aligned} 14\ V_1 + 6\ V_2 - 27 = 0 \\ 6\ V_1 + 8\ V_2 - 21 = 0 \end{aligned}\right\}$$

Solving for V_1 and V_2, we find, as before:

$$V_1 = \frac{270}{228} \text{ and } V_2 = \frac{396}{228}.$$

Fig. 18 shows a one-story bent of the viaduct, loaded with a concentrated wind pressure of 13.4 tons at the top and a uniform wind pressure of 30 lbs. per square foot on the column.

The influence of the bending forces alone will be examined.

Separating the bent by a cut "$\alpha - \beta$" at the end of the header,

Mr. Grimm. and applying at the cut and at each fragment, but in opposite directions, a moment M, a transverse force V and an axial force H, we obtain the following three equations:

$$\left.\begin{array}{rrrrrrr} -603 & V+ & 126\,M+ & 1942 & H= & 13\,515 \\ +9922.6 & V- & 603\,M- & 9253 & H= & 47\,010 \\ -9253 & V+1\,942 & M+40\,339.5 & H= & 282\,059 \end{array}\right\}$$

The values of the unknowns are:

$V = 15.8$ tons; $H = 7.07$ tons; $M = 73.7$ foot-tons.

The point of contraflexure, in this particular case, is moved a little distance out of the middle of the header, and this fact is due alone to the influence of a uniformly distributed wind pressure.

If the bent is influenced solely by the concentrated wind pressure of 13.4 tons, the three equations will be:

$$\left.\begin{array}{rrrrrrr} -603 & V+ & 126.5\,M+ & 1942 & H= & 13\,011 \\ +9922.6 & V- & 603\,M- & 9253 & H= & 45\,046 \\ -9253 & V+1\,942 & M+40\,339.5 & H= & 270\,275 \end{array}\right\}$$

and the unknowns are

$V = 15.2$ tons; $H = 6.7$ tons; $M = 72.4$ foot-tons.

These are the same results as given in the paper. The point of contraflexure of the header is at its middle, and the compression of the header equals one-half of the wind load. For all practical purposes, the concentrated wind pressure will be found to be sufficiently exact.

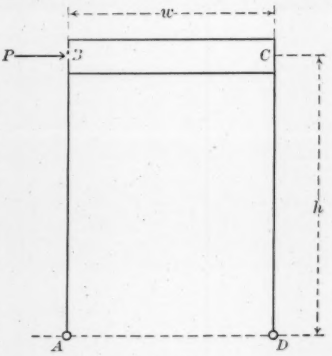

FIG. 22.

The very small influence of axial forces, with the consequent shifting of the points of contraflexure of the cross-pieces out of their middle is shown by the next example (see Fig. 22). $A\ B\ C\ D$ represents a hinged bent with a cross-strut or a floor beam rigidly riveted between the tops of the columns, and acted upon by a single concentrated force P at the top. This bent is singly statically indeterminate.

Let w = the width of the bent;
h = " height " " "
I = " moment of inertia of $A\ B$ and $C\ D$;
I_2 = " " " " " $B\ C$;
A = " area of $B\ C$;
M = any bending moment;
F = " axial force;
E = the constant modulus of elasticity;
H = the thrust at the hinge D (unknown).

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The condition equation is

$$\int \frac{M}{E\,I}\,\frac{\delta M}{\delta H}\,d\,x + \int \frac{F}{E\,A}\,\frac{\delta F}{\delta H}\,d\,x = 0.$$

The writer's investigation gave the following result:

$$H = P\,\frac{\frac{1}{2} + \frac{h\,I_1}{3\,w\,I_1}}{1 + \frac{2\,h\,I_2}{3\,w\,I_1} + \frac{I_2}{A\,h_2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The third term in the denominator is due to the influence of axial forces, and, neglecting this influence, Equation (3) becomes

$$H = P\,\frac{\frac{1}{2} + \frac{h\,I_2}{3\,w\,I_1}}{1 + \frac{2\,h\,I_2}{3\,w\,I_1}} = \frac{P}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Equation (3) shows:

1.—That the influence of axial forces is small on the final results, and

2.—That by their action the point of contraflexure is but slightly moved out of the middle of the cross-pieces.

Equation (4) shows that the point of contraflexure of the cross-piece is exactly at its middle, if axial forces are neglected. A two-story hinged bent has four unknowns, but if we know in advance that the consideration of bending forces alone, together with concentrated exterior loadings at the panel points, leads to the position of the points of contraflexure of the cross-pieces in their middle, we can treat such a bent as a singly statically indeterminate structure.

Of course, causes other than the axial forces, as also a change in the exterior loads, will cause the points of contraflexure to move out of the middle of the cross-pieces. Investigations like the foregoing will lead to considerations of trusses with diagonals, be they cantilevers or not.

Much more could be written, if time permitted, but it is believed that enough light has been thrown on the subject of the paper to show that it cannot be disposed of by mere opinions or assertions, without presenting argument or evidence.

In conclusion, attention is called to a comparison of the analytical results with those obtained graphically, which latter are quite useful in many cases, particularly so, if a design is governed by strong specifications.

The arbitrary assumption in the graphical method, which places the points of contraflexure of the columns in their middle, deprives the problem of its last trace of difficulty, and lowers the mathematical treatment to the level of a kindergarten method.

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No. 900.

FLOW IN THE SEWERS OF THE NORTH METROPOLITAN SEWERAGE SYSTEM OF MASSACHUSETTS.

By Theodore Horton, Assoc. M. Am. Soc. C. E.

Presented March 6th, 1901.

WITH DISCUSSION.

During the past few years several series of current meter observations have been carried on at intervals in the Metropolitan Sewers in the vicinity of Boston, Mass., for the purpose, first, of obtaining a suitable means for recording the daily flow of sewage pumped at the various pumping stations, and, secondly, of studying the changes in the carrying capacities of the sewers, due to the deterioration of the internal surfaces. Data for the flow of sewage in channels of large size are somewhat scarce, and, therefore, the writer feels justified in presenting a short description of a portion of the work which he thinks may be of interest, and perhaps of service, to many.

The work was done under the direction of William M. Brown, Jr., M. Am. Soc. C. E., Chief Engineer of the Metropolitan Sewerage Commission, and was carried out under the immediate charge of the writer.

The portion of the work herein described consisted of three series of observations made in the channels of the North Metropolitan system, below the Charlestown and the East Boston pumping stations. The first series was made in 1896, soon after the pumping stations and systems at these points had been put in operation. The next series was made in 1897, after these channels had been in operation about two years, during which time ample opportunities for changes in the carrying capacities had taken place. The third or last series took place in 1900, and was made just previous to the tests of the additional pumping plants recently installed at these stations, in order that a suitable rating of the channels could be made for obtaining the quantity of sewage discharged by the pumps during these tests.

The methods used in carrying out these sets of experiments were, with slight modifications, the same in all cases. An Ellis current meter, with four vanes, and provided with an electrical recording instrument, was used. Previous to each series of observations, it was carefully rated over a course varying in length from 500 to 1 000 ft., and for varying speeds. From these tests a rating chart was constructed for use in the calculations.

The points selected for carrying out these observations were at manholes located some distance below the pumping stations, where the flow was free from any disturbing influence of the pumps. The points were about 800 ft. below the stations, in each case, and were far removed from any changes in alignment, cross-section or grade of the sewer. Below the East Boston pumping station the cross-section of the sewer is a 9-ft. circle of 12-in. brick work, cement-washed, with a hydraulic gradient of 1:3 000. Only one small local connection enters this stretch of sewer. No changes in grade occur within a distance of 7 000 ft. below the pumping station. At a point 2 000 ft. below the pumping station, there is a change from a circular section to a horse-shoe section of the same equivalent area. This section continues for a distance of 2 000 ft. and then returns to a circular section. Below the Charlestown pumping station, the cross-section of the sewer is 6 ft. x 6 ft. 8 ins., basket-handle, of 8-in. brickwork, cement-washed, with a hydraulic gradient of 1:2 000. The cross-section and grade are uniform for a distance of about 5 000 ft. below the pumping station, and no local connections enter the sewer within this distance.

In order to determine whether the change of section in the East

Boston channel would have any appreciable effect on the flow in the circular section above it, simultaneous observations of depth were made along this channel for various rates of pumping. These measurements showed that no sensible variation of depth of flow occurred, except near the point where the cross-section changes; and that, for the stretch of channel where the observations were carried on, the flow line and invert were sensibly parallel. Such a study, below the Charlestown pumping station, was not deemed necessary, owing to the greater length of uniform section and grade.

Before operations in the channel were commenced, careful gaugings of the cross-sections were made; graduated straight-edges, provided with plumb-bobs, were inserted; suitable gauges erected for observing the depth of flow in the channel; and a platform hung in convenient position from which to operate the meter.

For the purpose of determining more exactly the position of the current meter while taking observations, a modification of the usual method of using this form of Ellis meter was adopted. The meter was run upon a $\frac{3}{4}$-in. graduated steel rod, in such a manner that it could be clamped at any desired position along the rod. In taking a reading the meter was immersed, one end of the rod was placed in the current, firmly against the side of the channel, and the other end held firmly in the hand and centered at a definite point on the straight-edge—the straight-edge being secured in a horizontal position across the channel and slightly above its center. In this position the meter rotated freely upon the rod, which in turn could be held in any direction in a plane at right angles to the axis of the channel. The position of the meter could thus be located in the following manner: By a distance along the steel rod from the wetted perimeter in a radial line from the central point on the straight-edge; a vertical position in reference to the horizontal straight-edge, measured vertically by a graduated wire attached to the central axis of the meter; a horizontal distance measured along the graduated straight-edge in reference to the vertical diameter of the channel. It is thus seen that the position of the meter was fixed by three measurements, of which only two were necessary, the third one affording a check. In fact, too much attention cannot be given to this point, especially with channels of moderate size, and in which the variations of the velocity in the cross-sections may be great.

In taking the observations, two men were stationed on the platform, one to hold the meter and rod in the desired position, the other to observe the vertical and horizontal position with reference to the straight-edge. The positions of the meter were usually selected in advance. Beginning with the meter at a distance of 0.25 ft. from the wetted perimeter, individual velocity readings were taken successively at each foot and half-foot to the right and left of the vertical diameter of the channel. The meter was then run up the rod to a distance of 0.5 ft. from the wetted perimeter, and a second set of readings taken to the right and left of the vertical diameter, and so on.

On the surface of the ground a third man recorded the revolutions of the meter by means of a stop-watch and electrical clock register. As an Ellis meter of this type registers only half revolutions, and as the intervals for single readings were sometimes necessarily short, dial readings of the electrical instrument were not used, but, instead, the revolutions were counted directly by means of the stop-watch and by listening to the click of the electrical register. A fraction of a revolution could thus be easily estimated. The interval for each single observation was usually 1 minute, but ranged from 30 seconds to 2 minutes, according to the time available for a complete cross-sectional measurement. If run for a longer interval than 2 minutes, it was found that the interferences from clogging and the interruptions of the electrical recording apparatus became so numerous as to interfere seriously with the measurements.

Between each of the individual readings the meter was taken out and carefully inspected to determine whether any foreign matter had lodged upon it in such a way as to affect its correct registration. The frequency with which such small interferences occurred seemed a serious draw-back at first, but by selecting a suitable time interval, by careful inspection of the meter, by noting carefully any deviation in the initial rate at which the meter registered when first immersed, and by comparing the consistency in the final results, errors from such sources were easily determined. All suspicious readings were, of course, repeated, and frequent check readings were made during a cross-sectional measurement.

Before starting a cross-sectional determination for any given depth, the pumping at the station was maintained at a uniform rate until the flow had become constant and free from any possibility of accelera-

tion or back-water. This was easily determined by frequent gauge readings at the observation manhole, and when uniform flow was once established it was maintained, within close limits, until the desired number of meter readings had been taken. These conditions were easily maintained, except when the pumping was at a rate different from the normal rate of flow in the sewer, in which case the pumps were working against a variable lift and required constant changes in the speed in order to maintain a constant discharge. During observations with low flows, the sewage stored up in the suction sewer was utilized in furnishing an increased discharge for high rates of flow.

In working up the results of these measurements, graphical methods were used entirely. Cross-sections of each channel, to suitable scale, were plotted from the actual gaugings taken in the sewer at the observation manhole.

A separate sheet of cross-section paper was devoted to each sectional measurement, and upon it were plotted the sewer section, the straight-edge, the average water line, and the various positions of the current meter during the sectional measurement, with the corresponding velocities computed from the field notes and rating chart. Vertical and horizontal lines were then drawn so as to include as many positions of the meter as possible, and, by means of these, suitable horizontal and vertical velocity curves were constructed. The method previously described for placing the meter in position, during the observations, brought most of the positions of the meter on vertical lines at even distances on each side of the vertical diameter. This was not the case with the horizontal lines, although the points lie sufficiently close to the selected horizontal lines to approximate closely the desired purpose. From these horizontal and vertical velocity curves, points on the cross-section were obtained, through which iso-velocity curves could be drawn and the fluid prism calculated. It will be seen that such a set of auxiliary horizontal and vertical curves gives independent locations of points for the iso-velocity curves, and, when there was any apparent disagreement, the mean location of the point was adopted.

This method of developing a sectional measurement is shown in detail in Fig. 1, the upper set of diagrams corresponding to No. I of the Charlestown series of 1900, and the lower set to No. IV of the East Boston series of 1900.

METROPOLITAN SEWERAGE, SHOWING VERTICAL AND HORIZONTAL VELOCITY CURVES, ISO-VELOCITY CURVES AND CURVE OF INTERNAL FRICTION.

CHARLESTOWN CHANNEL; 6′ 0″ X 6′ 8″, BASKET-HANDLE SECTION; GRADE 1:2000.

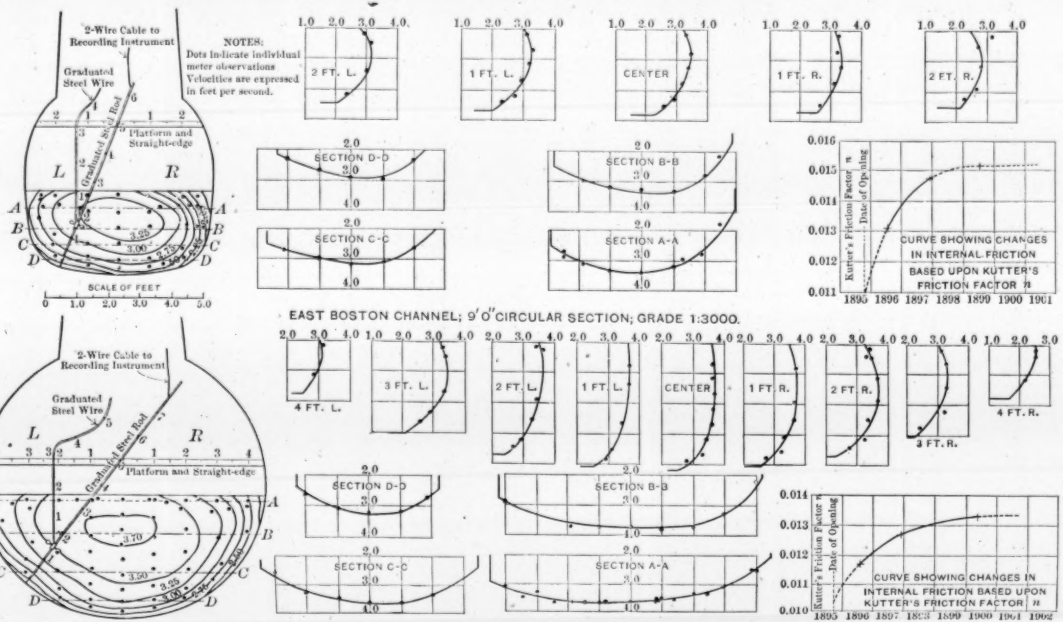

FIG. 1.

The volumes of the fluid prisms were computed according to the prismoidal formula, and the results were tabulated and compared with the Kutter formula according to values of n. From a comparison of these values, a suitable value of n was then selected, from which, with the geometrical elements of the cross-section, a discharge table could be constructed for use at each pumping station.

The results of these observations, arranged in series according to the periods at which the observations were made, are shown in Table No. 1.

TABLE No. 1.

No.	Depth.	Q. in cubic feet per second.	Mean velocity.	Hydraulic radius.	C.	n.
Series of July, 1896. Charlestown Pumping Station.						
I.....	1.02	8.60	1.99	0.688	107	0.0129
II.....	1.44	16.59	2.46	0.958	112	0.0131
III.....	1.91	26.81	2.82	1.187	115	0.0132
IV.....	2.40	38.82	3.13	1.387	118	0.0133
V.....	2.89	52.90	3.44	1.539	124	0.0130
Series of July, 1896. East Boston Pumping Station.						
I.	1.02	6.10	1.58	0.619	110	0.0122
II.....	1.52	15.67	2.21	0.928	126	0.0117
III.....	2.04	29.40	2.70	1.208	134	0.0116
IV.....	2.45	42.18	3.03	1.408	139	0.0115
V.....	3.16	69.50	3.48	1.830	141	0.0117
VI.....	3.75	94.60	3.73	1.999	145	0.0116
VII...	4.62	138.00	4.18	2.309	150	0.0115
Series of December, 1897. Charlestown Pumping Station.						
I.....	2.91	45.67	2.97	1.540	107	0.0149
II.....	3.29	56.14	3.16	1.650	111	0.0147
Series of November, 1897. East Boston Pumping Station.						
I.....	2.15	30.13	2.55	1.280	123	0.0126
II.....	2.74	47.75	2.90	1.560	127	0.0127
III.....	3.19	62.05	3.06	1.702	126	0.0129
IV.....	3.20	64.82	3.18	1.771	131	0.0126
Series of June, 1900. Charlestown Pumping Station.						
I.....	2.20	30.82	2.66	1.342	102	0.0151
II.....	2.78	41.39	2.86	1.508	104	0.0152
III.....	3.26	52.96	3.04	1.645	106	0.0152
Series of April, 1900. East Boston Pumping Station.						
I.....	1.99	24.96	2.38	1.120	119	0.0130
II.....	2.83	48.26	2.82	1.606	121	0.0132
III.....	3.64	76.78	3.16	1.952	124	0.0133
IV.....	4.18	95.84	3.30	2.130	124	0.0136

In July, 1896, the system at these points had been in operation about 10 months, and the masonry was in almost its original condition. The sides of each sewer, above and below the average water line, were covered with a thin coating of grease of a leaden color, and supported an organic growth—probably of a fungus nature, as probably no light ever entered the sewer, except temporarily.

The condition of the sewers in November and December, 1897, after a continuous service of 2 years and 2 months, was similar to that at the time of the 1896 observations. There were slight incrustations at frequent intervals, due to barely visible seepage of ground-water, and the growth, though not much heavier, covered a greater area on the sides of the sewers.

The condition of the channels in April and June, 1900, after a continued use of 3 years and 9 months, was not much different from that which obtained during the 1897 observations. The incrustation seemed to be slightly greater, but the organic growth did not show any marked increase over that present in 1897.

A discussion of any observations such as these should be made with caution, and the following remarks are offered more as suggestions than as the writer's views. It will be seen, first, from a hydraulic standpoint, that the results of the individual observations of any one series show a slight lack of uniformity, and, when compared according to values of n, do not arrange themselves in any definite order. This may be due to slight inaccuracies, from various sources, in dealing with a fluid of the nature of sewage; or to relative variations in the condition of the internal surfaces at the different depths of flow; or, perhaps, to the variation in the composition of the sewage itself. However this may be, it is found that if the different series themselves be compared, these slight differences fade in importance, and the results show a much greater uniformity. This is illustrated more clearly in Table No. 2, which gives the average values of n, in the three series, for each channel, arranged in the order of time.

TABLE No. 2.—VALUES OF n FOR THE CHARLESTOWN AND EAST BOSTON CHANNELS.

	1896.	1897.	1900.
Charlestown channel	0.0131	0.0148	0.0152
East Boston channel	0.0117	0.0127	0.0133

If the results in Table No. 2 be plotted, as shown in the diagrams on Fig. 1, with the horizontal scale representing time intervals from the date of opening of each system, and the vertical scale representing the tabulated values of n, and a curve be sketched through them, we might generalize as follows:

(1) Both curves are similar; the Charlestown channel curve being the steeper, and indicating a greater change in internal friction.

(2) Both curves show a sharp rise near the origin, indicating that the greatest change in internal surface took place soon after the channels were put in operation.

(3) That if the curves be projected back to the vertical axis, they show roughly the same initial coefficient of friction n, between 0.010 and 0.011, the Charlestown channel giving slightly the higher value.

In comparing these changes in the values of n with the actual condition of the channels, it should be kept in mind that the East Boston channel is of 3 ft. greater diameter than the Charlestown channel, that the invert of the East Boston channel is approximately 3 ft. above mean low water, while the Charlestown channel is 4 ft. below mean low water, and that the East Boston channel receives relatively less storm water than the Charlestown channel, and is, consequently, subject to less fluctuation of water surface. The importance of this last influence is evident from the fact that the deposit of both grease and organic growth appeared in greater abundance on the sides of the channels, and was greatest near the line of average flow. On the bottom of the channel there was practically no deposit; resulting, no doubt, from the scouring action of sand and other heavy particles transported along the invert by the sewage. This last feature is by no means novel, and has frequently been observed, though to a less extent, in water-supply conduits.

The effect of the density of the sewage upon the carrying capacity of these channels appears to be slight, in view of the fact that the observations were made under all the varying conditions of storm and dry-weather flow. The possible effect of cleaning or scraping, however, might be much greater, but, at this date, no cleaning of any sort has taken place in these channels. Such questions, with others of equal interest, will, it is hoped, have more light thrown upon them by the future continuance of this work.

DISCUSSION.

Mr. Hering. RUDOLPH HERING, M. Am. Soc. C. E.—Papers of this nature are of special value because they add to the fund of knowledge by giving the results of experiments made on a large scale in a field where there is a dearth of exact information. In very few sewers has the flow been carefully gauged, and, for practical purposes, data derived from the flow of clean water are generally used. It is, therefore, especially gratifying to see a paper on this subject. The information it contains seems to be reliable, and, if so, it is certainly useful.

Presumably, the meter was rated in clean water instead of sewage, and, to avoid clogging during the gauging, the meter is said to have been withdrawn every two minutes. The chief trouble in gauging sewage is caused by the suspended matter catching in the wheel and retarding its motion. The speaker would like to know how far this feature was considered, and how far the results obtained can be applied to other cases.

The lining of the sewers was apparently the same in both instances; namely, cement-washed brickwork. Yet the coefficients of resistance were different, apparently due to the fact that there was a greater rise and fall in the water level in one case than in the other, consequently a greater accumulation of matter adhering to the sides of the sewer and a greater resistance to the flow. The importance of the adhering matter as affecting the velocity is, therefore, clearly indicated.

The hydraulic slope seems to have been assumed as being the same as the slope of the invert. What effect was, then, caused when the section changed, and was any observation taken of this effect? Calculating the discharge by the prismoidal method is unusual but very good.

The small diagrams show the changes of the internal friction with age. A similar change takes place even with clean water, and in open channels as well as in pipes. It indicates the usefulness and rationale of having a coefficient depending substantially on surface friction alone, so that this change may be separately and properly accounted for in the formula used. In extending the curve from the known points back to the time when the sewer was built, the speaker thinks that the curve is dropped too low. New brickwork under similar conditions does not, as a rule, give so low a coefficient, but generally ranges from 0.011 to 0.012, instead of from 0.0105 to 0.011. Therefore, it would seem better to have drawn a tangent than a curve from the last point of observation to the ordinate representing the condition of the sewer when new.

In a Colorado city the speaker noticed at one time so large an accumulation of slimy growth attached to a pipe that when loosened by a flush it caused a complete stoppage. In Philadelphia, near Girard Bridge, he noticed in a 4-ft. sewer such a coating, consisting

Mr. Hering. of mycelium, that was more than 1 in. thick at the soffit. It was soft and gelatinous.

Mr. Smith. J. WALDO SMITH, Assoc. M. Am. Soc. C. E.—It is rather gratifying to see how well the results obtained by Mr. Horton seem to agree with the results of the flow of water in large riveted steel conduits. That is to say, the first deterioration is of considerable amount and very rapid, and afterward the deterioration seems to be much slower, but continues for years. The speaker would be glad to learn just why a thin coating of slime should so greatly increase the friction. The fact is undisputed that the loss in carrying capacity due to this is very great, and in some cases under the speaker's immediate notice this loss has in four years amounted to over 20% of the original carrying capacity; one-half to two-thirds of this loss probably occurring in the first year. Mr. Horton's general results seem to show that the same is true of a cement-washed brick sewer.

Mr. Tillson. GEORGE W. TILLSON, M. Am. Soc. C. E.—Mr. Hering's remarks remind the writer of an experience he had in Omaha, of practically the same nature. The sewer system originally built there in 1881 or 1882 was made up wholly of vitrified pipe, the laterals all being 6-in. They were short, those first built extending, probably, not more than half a mile; the lower two-thirds having a grade of about 6 ins. in 100 ft., and the remainder anywhere from 4 to 8 ft. in 100 ft. The upper end was provided in every case with a flush-tank, but these worked very irregularly, and, probably for weeks, not at all. The system was built entirely without manholes on the laterals, and, after the sewers had been in use perhaps 1½ or 2 years, the laterals on which the greatest number of connections had been made became clogged, and gave a great deal of trouble. It was necessary to dig down and open the pipe, and then this growth that Mr. Hering speaks of was discovered. It extended practically through the whole length of the sewer, where the grade was light. The part where the residences were had very few connections. The only way the sewers could be cleared out was by digging down at intervals of perhaps 250 or 300 ft., putting a float through and then dragging a ball after it. A great deal of attention was required to keep the sewers clear, and, eventually, manholes were built every 364 ft., and the sewers were kept clean by flushing, until the connections became so numerous that it was necessary to rebuild the sewers and put in 18-in. pipes, after which there was no trouble at all. But it does seem that, in these small pipe sewers, unless there is sufficient flush to keep them approximately clean, there will be this growth on the inside of the pipes, which will materially reduce and almost entirely destroy their efficiency.

Regular flushing, however, will keep them clean. The sewers which were built afterward, where the flow was slight and where there were better regulated flush-tanks, gave no trouble at all.

The growth referred to was a sort of fungus of the streamer style, perhaps 5 or 6 ins. long, and very bulky when it was first taken out, but after exposure to the air it would dry up and almost disappear. Mr. Tillson.

G. C. WHIPPLE, Assoc. M. Am. Soc. C. E.—The speaker cannot, of course, tell just what organisms were growing upon the walls of this particular sewer without making an examination, but, in order that it may be placed as a matter of record in connection with the paper, he will be very glad to make an examination of any material which the author may send him, and transmit the results by letter. Mr. Whipple.

The organisms which grow in sewers are very different from those which grow in water pipes. Speaking broadly, it may be said that those found in water-pipes belong to the animal kingdom, while those found in sewers are vegetable forms of a very low type. In water-mains we have the fresh-water sponges and several genera of *Bryozoa*, such as *Plumatella*, *Fredericella*, *Paludicella*, etc.; organisms which require oxygen, and which, for their food, depend upon the microscopic organisms found in water. It has been found that a large amount of suspended matter, however, is prejudical to their growth, as it clogs up their minute pores. For this reason sponges and *Polyzoa* are found upon the top and sides of pipes rather than upon the bottom. It is easy to see, therefore, how the suspended particles and greasy substances found in sewage would tend to prevent these organisms from becoming fixed upon the walls of sewers. So far as the speaker has examined deposits in sewers he has never found any of the sponges or *Bryozoa* (*Polyzoa*) present.

The organisms which most abound upon the walls of sewers, and upon the sides of tanks through which sewage passes, belong to the fungi—plants which have no chlorophyll and no starch. Lacking these, they are unable to assimilate inorganic matter, and, consequently, have a saprophytic or a parasitic existence; that is, they live upon dead organic matter or upon some living host. Most of the fungi consist of two parts, the mycelium and the fruit. The mycelium is the vegetative portion of the plant, and is often a mass of delicate, jointed, branched, colorless filaments intertwined to form a cottony or felty layer. It is these mycelia which grow luxuriantly upon the walls of receptacles containing sewage. The fruit consists of certain terminal filaments erected from the general mass and bearing spore-cells of various kinds. It is chiefly by differences in the method of fruiting that the different fungi are distinguished from each other. Hence, when only the mycelium is to be found, it is difficult and often impossible to identify the specimen. Of those fungi most commonly found in sewers, may be mentioned: *Leptothrix*, *Cladothrix*, *Beggiatoa*, *Mucor*, *Penicellium*, *Aspergillus*, *Saprolegnia*, *Leptomitus*, *Achlya*, *Saccharomyces*, etc.

*On March 19th, 1901, the writer received from the author three

*The remainder of this discussion was received from Mr. Whipple by letter.

Mr. Whipple. samples of material taken from the walls of the sewer. These were examined microscopically with the following results:

Sample No. 1.—"Coating scraped from sides of sewer. When in place it consists of a layer of black material $\frac{1}{8}$ to $\frac{1}{4}$ in. thick, attached to the wall, and this is covered with a thin gray coating, which gives the gray appearance to the sewer."

The black mass is essentially a mass of bacteria interlaced with threads of *Beggiatoa* and mold hyphæ. The black color is due chiefly to sulphur compounds, held, perhaps, by the *Beggiatoa*. Many of the bacteria were in the zoöglœa state, but many cells were seen singly, in pairs and in chains. *Spirilla* were common. Many yeast-cells (*Saccharomyces cerevisiæ*) were present, and a few grains of starch and globules of fat. The gray surface coating consists chiefly of interlaced mycelium threads. These are in part mold hyphæ and in part threads of *Leptomitus*. One branch of *Penicillium* was observed in fruit. A few microscopic organisms, such as *Trachelomonas*, *Melosira*, *Rotifer*, etc., were found entangled in the mass.

Sample No. 2.—"Sample of the gray coating separated from the surface of the black material mentioned under Sample No. 1. It was about $\frac{1}{32}$ in. thick."

As mentioned above it consisted of interlaced mycelium threads, with yeast cells, starch grains and *Beggiatoa* filaments, with bacteria covering all. In this sample there were many ciliated infusoria, such as *Glaucoma*, *Halteria*, *Trachelocerea*, *Paramœcium*, etc.

Sample No. 3.—"Sample of streamers which are attached to the coating on the sides of the sewer, and which float out into the stream."

These consisted chiefly of *Leptomitus*, the common "sewage fungus." The hyphæ of *Leptomitus* are long, cylindrical, branched, and deeply constricted at intervals and at the point of branching. Near the constriction there is usually a globular body, like an oil-globule. The grayish protoplasm generally occurs as evenly distributed lumps, but sometimes it is arranged in concentric masses. The zoospores are formed in club-shaped terminal sporangia. A few of these sporangia were observed in the sample. The hyphæ are practically colorless.

Leptomitus often develops in great masses upon the walls of sewers and on the banks of polluted streams. The writer has several times seen it forming layers from 5 to 8 ins. thick, and it is easy to believe that small pipes may be almost completely choked by it. Apparently, it grows best in the absence of oxygen, and in this respect it agrees with some of the *Schizomycetes*.

Sample No. 4.—"General deposit from material on sides of sewer."

The bulk of this deposit consisted of a mass of small worms embedded in amorphous matter of bacterial origin. Mold hyphæ were abundant, but there was little *Leptomitus*. Several fragments of paper and a small piece of cloth were found, besides several fibers of wool, pieces of straw, etc. This sample contained more microscopic organisms than the others.

THEODORE HORTON, Assoc. M. Am. Soc. C. E. (by letter).—The discussions on this paper have been more in the nature of additions than of differences of opinion, hence the writer feels that he can add little of interest to what has already been said. A few points were brought up informally, however, which have not yet been presented and which may be worthy of mention. Mr. Horton.

The first of these relates to the method, adopted in these measurements, of operating the current meter from a fixed platform in the sewer, the locations of the meter being obtained by a system of rectangular co-ordinates, as compared with other methods adopted successfully elsewhere, notably in the Sudbury Aqueduct experiments, in which the meter was operated from a framework in an observation manhole, the locations of the meter being obtained by a system of polar co-ordinates. The choice, in the present case, was dependent upon the nature of the fluid conveyed in the channel, for with sewage the suspended matter was of such a nature and in such quantities as to necessitate frequent inspections and occasional cleaning of the current meter, in order to insure correct registration. The suspended matter occasionally met with in conduits conveying water is, on the contrary, usually of such a nature as to be readily screened, and any trouble from clogging of the meter is avoided. It was these frequent withdrawals of the meter during a sectional measurement which, it was thought, were more easily and quickly made by the method of operation adopted than by the other method which, though very efficient, was slightly more complicated. Incidentally, also, the operation of a meter from a framework involves the necessity of a certain amount of working space in a manhole. In the present case the space was somewhat limited and might have proved inadequate.

A second point, and one which bears slightly upon the same question, was in regard to operating the meter from a pivotal position near the center of the cross-section. This is somewhat important, considering the type of meter used in these measurements, for with an Ellis meter the motion depends upon the difference of pressure on the two sides of the cup-shaped vanes. If, then, the meter is placed in a stream where the filaments are parallel and travel with the same velocity, we have the conditions equivalent to those under which the meter is rated in still water. If, however, the filaments which impinge against the vanes move with varying velocities, an erroneous mean velocity for the central point of the meter is recorded. This influence, of course, is inappreciable in streams of large cross-section where the variations of velocity in the cross-section are very gradual, but in closed channels of moderate size these variations are more sudden, especially near the periphery, and the possibility of error is consequently greater. By operating the meter from a pivotal position, however, as was done in these measurements, the plane of the revolv-

Mr. Horton. ing vanes is, for nearly every position of the meter in the cross-section, tangent to the iso-velocity curves, so that the velocity recorded by the meter in any such position is, so far as this one influence is concerned, a correct one. Near the surface of the stream, where the iso-velocity curves are nearly horizontal, this influence was avoided by holding the meter in a horizontal position.

Many questions of interest from a biological standpoint suggest themselves, especially if the composition of sewage is contrasted with that of fresh water. Normally, sewage has an abundance of organic food supply and little or no dissolved oxygen. It abounds in organisms of a fungous nature which develop without the direct action of sunlight, and many of them without dissolved oxygen. Fresh water, on the contrary, has relatively a small amount of organic food supply and usually an abundance of dissolved oxygen. It supports normally a limited growth of organisms. Such organisms are almost wholly green plants which are dependent upon the direct action of sunlight for their development. These, when they reach the closed dark water-supply conduits, probably cease to develop.

In channels conveying sewage there is a marked increase in the filamentous growths on the sides of the channels during the summer months, due, no doubt, to the increased temperature of the sewage at this season. This increase of growth was quite noticeable in some portions of the Metropolitan sewers, and many instances have been cited where such growths have been sufficiently abundant to completely close the channel. In water-supply conduits this seasonal development of organisms is probably also active to a degree, but is probably much diminished by the fact that the organisms normally conveyed into the conduits are, owing to the darkness, in an environment unfitted for any further development. In fact, the troubles so often caused by growths in such conduits are usually due to abnormal conditions, and are rarely found in well-maintained water supplies. The main point, however, is that these growths, when they do occur, cause increased internal friction and a consequent decrease in the carrying capacity of the channels.

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No. 901.

STEEL-CONCRETE CONSTRUCTION.

An Informal Discussion at the Annual Convention, June 26th, 1901.

SUBJECT FOR DISCUSSION:

"Steel-Concrete Construction.
"What stress in tension and compression should be allowed in concrete?
"What is the proper modulus of elasticity of concrete?
"In Steel-Concrete Arches:
"(1) What should be the ratio of steel section to concrete section, and what is the best form and disposition of the former?
"(2) What consideration should be given to temperature changes and consequent stresses?
"(3) What are the best proportions for concrete, and what is the best method of placing it?"

By Messrs. R. S. BUCK, L. S. MOISSEIFF, J. R. WORCESTER, H. S. JACOBY, LEONARD C. WASON, A. L. JOHNSON, WILLIAM PARKER and WILLIAM B. FULLER.

Mr. Buck.

R. S. BUCK, M. Am. Soc. C. E.—There are still doubts in some quarters, not only reputable but excellent professionally, as to the thorough soundness of steel-concrete construction for arch bridges.

The extent to which there is just cause for such doubts, and whether or not the cause can be removed by revision of practice, it is hoped this discussion will assist in disclosing. The speaker himself does not hope to contribute more than the suggestion of a few points,

Mr. Buck. which, in his limited experience with the system, struck him as being in need of discussion and elucidation.

Steel-concrete construction in arches has, both in Europe and this country, passed from the experimental stage to that of general practical use.

Although there is still lacking the crucial test of time, the large number of these bridges now in service seems satisfactory, and there appear no reports of failure attributable to any inherent weakness of the system, despite some remarkably bold dealings with it.

If, as now appears reasonably certain, the system is not only here to stay, but is having its field of application steadily enlarged, it is surprising that there are not more working data on the subject, of certain scientific origin, in place of the variable "rule-of-thumb" methods which seem now to govern in American practice.

European engineers have given the subject much more careful study, and have conducted some very elaborate experiments to settle certain points. But the results of these experiments have reached us here only in very general terms, or else are buried in unfamiliar foreign publications, from which they have not yet been mined. That there is enough in the records of European practice, investigation and experiment, to frame a complete, reliable scientific code of procedure for this system, is doubtful; but there can be no doubt that they would contribute largely to this end.

Mr. Fr. von Emperger, on April 4th, 1894, read a paper entitled "The Development and Recent Improvement of Concrete-Iron Highway Bridges."* This was, perhaps, the first general introduction of the subject in this country, and with the exception of an article† by Edwin Thacher, M. Am. Soc. C. E., on the Paterson Bridge, this remains, so far as the speaker knows, the only attempt at analytical treatment of the system indulged in in this country. Mr. von Emperger's paper lacks many things, to render it of scientific or practical value, and was evidently intended rather to awaken interest in the possibilities of the system than to assist in establishing any laws whereby these possibilities could be confidently attained. It made known, generally, the fact that the Austrian Society of Engineers and Architects had conducted an elaborate and doubtless valuable series of experiments on metal-concrete arches, and stated that a report would be made on the same, probably at an early date. But this report has not, to the speaker's knowledge, either as a whole or in part, been published in English. It would be a most valuable contribution to our fund of knowledge if the report were so published, and the same supplemented by discussion and experiment along the lines of American practice.

* *Transactions*, Am. Soc. C. E., Vol. xxxi. p. 438.

† *Engineering News*, Sept. 1st, 1899.

As previously stated, there is a great deal of "rule-of-thumb" about American practice in this system, and its general acceptance cannot well come until such methods are replaced by others as carefully grounded as those governing the use of steel. Mr. Buck.

The engineers of this country have been slow in taking up this system, probably unduly so when its value is fairly considered. Its development has been largely in the hands of contractors and contracting engineers, with whom a vital point has been to bring it in competition with all-steel construction. In this, the speaker inclines to think, there has been rather an unfortunate disposition to reduce the cost unduly at the sacrifice of certainty, stability and durability. It is not possible to construct a steel-concrete arch that will compare in cost with an all-steel bridge and at the same time furnish the desired characteristics to be reasonably looked for in a masonry arch.

The steel-concrete bridge possesses sufficient merits, the speaker believes, when designed on conservative lines, to often reduce the question of first cost to a minor position; and when it cannot do this it is a good form of bridge to let alone.

The combination of concrete and steel unquestionably is one of great mutual benefit, especially to the concrete, but it is very questionable, to the speaker's mind, if this general proposition warrants the high unit stresses allowed on the strength of certain combinations.

On steel 20 000 lbs. per square inch; 500 lbs. per square inch compression on concrete, without allowance for temperature, which may become 1 000 lbs. per square inch under the same rules for fixing sections where a reasonable allowance for temperature is made; 50 to 100 lbs. per square inch in tension in concrete without consideration of temperature, and which, considering temperature, may become 200 lbs. per square inch; are allowances that may appear to meet demands, but, the speaker believes, are hardly justifiable.

The freak bridges built for experimental purposes or for pedestrian traffic in Europe, some of which Mr. von Emperger mentions in his paper, and the fact that these have stood the test of a few years, seem to warrant too much imposition on the system.

An arch of 132 ft. span, $14\frac{3}{4}$ ft. rise and $9\frac{7}{8}$ in. crown thickness (one of those mentioned by Mr. von Emperger, and claimed by him to be designed for a load of 200 lbs. per square foot) would seem to justify much, but until the speaker sees the 200 lbs. per square foot safely applied, and the test of years of service, he will not count such construction, or even a remote approach to it, good. Yet it is not improbable that in European practice such cases as this have to some extent influenced American practice, and in a measure justified thinning down sections when other justification was lacking. The differ-

Mr. Buck. ence between experiment and practice, and different kinds of service, demand very careful consideration. Concrete, despite its versatility and endurance, cannot be relied upon with the same confidence as steel, any more than can any other form of masonry, especially where tension is to be dealt with.

It is rather a striking exemplification of the uncertainty of our knowledge of the action of concrete in conjunction with steel, in arches or elsewhere, that there is such a large variation of opinion as to the correct modulus of elasticity of concrete. This ranges from 700 000 to over 3 000 000. This difference is of course largely due to the variation in composition of the concrete experimented with, but there should not be such a difference in the concrete suitable for arch rings—say Portland cement concrete in proportions of 1:2½:5. The modulus of elasticity of such concrete, within narrow limits, can and should be established much more definitely.

Directly contingent upon this modulus is the correct proportion of concrete to steel capable of taking calculable stresses in the arch ring, and in fact the entire theory of arch-stress calculation.

However, the correct combination of concrete and steel is probably quite as much dependent upon the thorough bonding of the two in order to secure unity of action as upon the capacity of the steel to pick up and carry to the abutments a specific amount of stress.

The correct function of the steel is rather to insure a thorough distribution of the strain in the concrete, and to prevent undue concentration of strain and consequent rupture at some point where there is an element of weakness akin to a nick or other flaw in a steel bar.

The steel should be so disposed as to bond the concrete in three directions, parallel or approximately so to the axis of the ring, normal to the soffit, and transversely normal to the axial plane of the ring; and so proportioned in each direction as to meet all possible demands upon it. Further, the units into which the steel is divided should be such as to insure its reaching all parts of the concrete with its influence.

In this respect the Monier system, while it apparently lacks the steel bonding normal to the soffit of the ring, possesses the advantage of fine division and thorough dissemination and bonding in the other two directions. The Melan system, with steel I-sections, bonds essentially normal and parallel to the intrados, but not transversely. The units of steel in this system are large, and the dissemination through the concrete not so thorough as in the Monier system. In fact, it may be fairly questioned whether this is sufficient to bring all the concrete properly within the range of influence of the steel.

The Thacher system, as originally adopted, comprising steel in the shape of flat bars parallel to the intrados and extrados of the ring, bonded the concrete only in one direction; it being assumed that the steel could do best service as flanges on these lines, and that no

other bonding was necessary. This system is susceptible of ready modification, which will secure bonding in all these directions, and the reduction in size and multiplication of number of units is practically limited to the space necessary for laying the concrete. Mr. Buck.

The treatment of temperature stresses, or at least their non-treatment, is rather an anomalous condition found in dealing with steel-concrete arches.

Mathematically calculated on clearly established proper lines, even with a limited range of about 30° Fahr., the resulting temperature stresses are decidedly startling, especially in flat arches. Although the correctness of the mathematical results is not questioned, it is often deemed expedient and safe to ignore them. The justification for this seems to be that the presence of the steel will prevent serious results if a crack should occur, and that many arches subjected to heavy temperature stresses stand without manifesting weakness. This reasoning is convenient, but hardly scientific or logical. This feature of the system, perhaps more than any other, is in need of thorough discussion.

The question of ingredients and the degree of wetness of the concrete in the arch ring is a very general one, but of much importance, and well worth discussion here.

Another point of importance is the method of laying in continuous rings from abutment to abutment or in large voussoirs. There is now a clear cut difference of opinion on this point, which is also well worth discussion.

Aside from the question of the correct theory governing the sections and proportions of a steel-concrete arch, the speaker inclines to the opinion that there is much room for improvement in the practical details of construction in American practice. Strength of centering and precautions against sinking; uniformity and sufficiency in ramming; careful bonding of fresh and set concrete, and care in securing monolithic construction; care in securing actual contact and adhesion between the concrete and the steel, are all points wherein practice can be materially improved. Ramming is perhaps one of the weakest points in steel-concrete construction, but the speaker looks with reviving hope to the pneumatic rammer which has recently been put upon the market. Some information as to its performance would be most welcome, and it is hard to believe that it will not be an immense improvement on the machine now in use, the cardinal principle of which seems to be to make as little impression on the concrete as possible without loss of its job.

L. S. Moisseiff, Assoc. M. Am. Soc. C. E. (by letter).—*Allowable Tensile and Compressive Stresses.*—With the usual proportions of Portland cement concrete of 1 : 2 : 4 and 1 : 3 : 5, and careful manipulation in making the concrete, a maximum compressive stress of 500 lbs. per Mr. Moisseiff.

Mr. Moisseiff. square inch and a maximum tensile stress of 60 lbs. per square inch may be allowed in beams and arches. A study of a great number of tests made in the United States, as well as in Europe, will show that these values are well within the limits of good engineering. It should be understood, however, that the greatest allowable unit stresses are really meant to be the maximum stresses which will occur in any section of the structure under any vertical or horizontal load, or other agency that can be reasonably expected to act on the same. For arches, stresses due to variations of temperature or to displacements of abutments should be treated as secondary stresses, their amounts should be determined, and they should be provided for within the greatest allowable unit stresses. Then only will the fixing of some limit of unit stress have a definite meaning. The secondary stresses will vary greatly with the proportions of the arches Generally, the flatter the arch, the greater will be the effects of temperature and settling; and for the flat arches the secondaries will show unexpectedly high values. Only by adding the secondary to the so-called primary stresses due to external loads can the maximum stresses acting in any part of the structure be determined. For the stresses so found, the structure should be proportioned by using the fixed maximum unit stress.

The purpose of the structure should also be taken into consideration. A light structure may be used for a highway bridge, with its light loads, but a railroad bridge, with its heavy impact, requires mass, in addition to strength, to absorb the vibrations due to rapidly moving trains. This may sometimes lead to greater thickness of arch than would be absolutely made necessary by the allowable unit stresses. Such increased thickness represents material well placed.

Modulus of Elasticity of Concrete.—The modulus of elasticity of concrete not reinforced by steel must be known in order to determine the secondary stresses. The basis of all formulas or graphical methods used to determine these stresses must be the elastic theory of arches. The famous tests made by the Austrian Society of Engineers and Architects have proved definitely that the elastic theory holds true for concrete arches as well as for any other kind of arches. This necessarily introduces the elastic behavior of the material, and with it the modulus of elasticity, into the determination of all stresses. But, whether the elastic theory or the static assumptions are used for finding the main stresses, the modulus of elasticity must be known, in order to determine the stresses due to displacements and to changes of temperature and volume.

For steel-concrete construction, the value of the modulus of the concrete is of prime importance. To these constructions, steel in the form of wire, bars or shapes is imbedded in a concrete matrix, forming, by the adhesion of the two constituents, a practically new material.

Mr. Moisseiff.

To have a clear understanding of the stresses in such a combination, the elastic behavior of the constituent parts under any stress must be thoroughly known. The modulus of elasticity represents the reciprocal of the elongation per unit stress and unit length, and its value indicates the story of the behavior of the material. In fact, the theory of composite structures, as it has been applied to steel-concrete, is directly dependent on the values of the moduli of elasticity of the two materials.

It has been shown by a wide range of experiments, especially by those made by Professor C. Bach, at the Royal Testing Laboratory at Stuttgart, that Hooke's law of the proportionality between stress and deformation does not hold true. It is very nearly true for steel and wrought iron, but for concrete the curve of elongations has a pronounced curvature, showing that the elongation increases in a higher degree than the stress. The modulus of elasticity of Portland cement concrete is then a variable, decreasing with the increase of unit stress. This characteristic of the elastic behavior of concrete is of importance in steel-concrete construction. It is evident that the more the concrete elongates per unit stress, the modulus of elasticity of the steel being practically constant, the more stress will be carried by the steel.

When the theory of composite structures was first applied to steel-concrete by Professor J. Melan he assumed the modulus of elasticity of Portland cement concrete to be 710 000 lbs. per square inch. This is about one-fortieth of the modulus of steel. This means that the stress in the steel will be forty times the stress in the surrounding concrete. But, even then, the steel will only be stressed in tension to $60 \times 40 =$ 2 400 lbs. per square inch. This is certainly a ridiculously small value for the tensile stress in steel. Since that time it has been found that the value $E = 700\,000$ lbs. for concrete is far below its actual value, and in recent years prominent engineers in this country have used $E =$ 1 400 000 lbs. This gives a ratio of 1 : 20. According to this assumption, the work of the steel is reduced by half.

A study of a great number of tests shows that the modulus of elasticity for Portland cement concrete made in the proportions between 1 : 2 : 4 and 1 : 3 : 5 is from 2 800 000 to 3 500 000 lbs. This makes the modulus of concrete one-tenth to one-eighth that of steel. This new value of the modulus for steel-concrete further reduces the proportion of the work done by the steel.

The question then arises, do we not pay too much for the little the steel does? This seems to be so, according to the accepted theory, but, fortunately, practice is here more correct than theory; in fact, too much should not have been expected of a theory based on pure assumptions. The observations made on numerous steel-concrete beams and arches show that the concrete is stressed considerably beyond any reasonable stress, or otherwise the steel is doing much

Mr. Moisseiff. more than is attributed to it by the composite-structure theory. Only a great number of careful experiments on steel-concrete combinations, in conditions as near as possible to those met in actual practice, can furnish reliable and sufficient data on which to base an adequate theory.

A series of careful experiments on the influence of imbedded metal on the properties of mortar and concrete was made in 1898 by the eminent French engineer, M. Considère, at the laboratory of the Ecole des Ponts et Chaussées. These tests have shown remarkable results. They have proved that concrete in combination with steel is able to elongate considerably more, without impairing its resisting value, than if not reinforced by steel. The same phenomenon has been observed in the experiments made at the same time for the X bridge at Mans, France, by the Chief Engineer, M. Harel de la Noë. This is also confirmed by observations made on arches and by computations made on observed deflections. It is out of place here to give this interesting subject more space.

The writer is at work on a paper, to be presented to the Society at an early date, on "The Theory of Steel-Concrete Structures," and this discussion is merely a brief abstract of its main features. Whatever the explanation of the different behavior of concrete when reinforced by steel from that when not reinforced, it is certain that the theory of concrete structures needs revision. Formulas based on the actual behavior of concrete and steel in combination should be deduced. These formulas will have to account rationally for the relative work done by the two materials. They will have to furnish means to evaluate the capacity of the steel and give the right proportion of steel to concrete.

Ratio of Steel Section to Concrete Section.—From the foregoing discussion it will be seen that no *a priori* ratio of steel section to concrete can be fixed. From some tentative computations, the results seem to require the section of the steel to be between 1 to 1.5% of that of the concrete. The steel should be as well distributed in the concrete as possible, reaching the greater part of the outside layers, and thus increasing the strengthening effect. It should not be nearer to the surface than 3 ins., and should be so disposed as to allow of easy and efficient tamping. It should be well tied together vertically and laterally, if only with very light members.

Temperature Changes and Consequent Stresses.—No stresses are so certain to take place as are the stresses due to changes of temperature and to displacement of abutments. The experience of French and German engineers, who have built numerous masonry arches, has been such as to lead them to the use of hinges. While it may be a disputed question whether or not hinges should be used in steel-concrete arches, it is certain that temperature stresses must be taken care of.

Mr. Moisseiff.

It has been the writer's experience in calculating temperature stresses in concrete arches, and it is known to him to be the experience of other engineers who have calculated the same, that the thermal stresses reach high values, even for small ranges of temperature.

It is quite difficult to determine what range of variation of temperature should be allowed for. A mass of masonry will certainly not change its temperature very rapidly, and its range of temperature will not exceed the range between mean cold and mean warm weather. A range of 40° Fahr. would probably be sufficient to provide for. Even this low range will cause thermal stresses high enough to require careful consideration.

Mr. Worcester.

J. R. WORCESTER, M. Am. Soc. C. E.—One statement by Mr. Buck, if correctly understood, that there could be no comparison between the cost of steel-concrete bridges and steel bridges, seems to the speaker to be true only when one factor, that is, the cost of the abutments for the steel bridges, is left out of consideration.

The speaker's experience is, that, in the case of a highway bridge of moderate span, a comparison between a steel-concrete bridge and one of steel with buckle-plate floor filled on top, will show that the steel-concrete bridge is considerably cheaper; and that where good foundations are easily obtained, if the total cost of bridge and abutments is included in the comparison, there is a fair chance for steel-concrete construction to compete with all-steel.

Mr. Jacoby.

HENRY S. JACOBY, Assoc. Am. Soc. C. E.—It seems now to be generally conceded that the theory of elasticity should be applied in designing steel-concrete and other masonry arches. While it has been customary to assume the moment of inertia of the cross-section to vary as the secant of the angle of inclination of the axis of parabolic arches, in order to simplify the determination of the horizontal thrust, no investigation of the relative effect on the design caused by this approximation has been published. It may, therefore, be of interest to call attention to the fact that some recent investigations show that the differences between the areas of cross-section obtained when the moment of inertia is assumed to vary in accordance with the law previously mentioned and those obtained when its true variation is considered are so small that they may be safely neglected in any design. As the difference in the labor involved in making the designs is so great, it is gratifying that the simpler method gives results which answer every practical purpose.

In regard to the amount of steel required, the speaker favors the plan adopted by Mr. Thacher, to use enough steel to resist the bending moments, and to let the concrete take the axial thrusts and the shear.

Attention is also called to the desirability of making some systematic observations of the deflections of existing steel-concrete arches

Mr. Jacoby. by those who may be conveniently located with respect to the larger arches in this country, such observations to extend throughout at least one entire year, so that definite data regarding their behavior under changes of temperature and live loading may be available.

Mr. Wason. LEONARD C. WASON, Assoc. M. Am. Soc. C. E.—The speaker has devoted his whole time for nearly eight years to the Ransome system of steel-concrete construction, and, as in fire-proof floors, the maximum stress in steel and concrete with minimum weight is necessary, especial attention has been given to the stresses in tension and in compression, the ratio of the steel section to the concrete section, and the proportions of cement and aggregates used. What follows is the result of personal experience, and, with the exception of the tests at the Watertown Arsenal, none of the data has ever appeared in print before.

Last winter the speaker made fourteen concrete beams for a student to test as thesis work at the Massachusetts Institute of Technology. The object of the tests was to determine accurately the strength of the combination, the position of the neutral axis of the beams, the maximum deflection at the first crack, and to see if the concrete cracked before the steel had reached its elastic limit. All beams were 12 ft. long, 8 ins. wide, and 12 ins. high, and were made as follows:

Beam.	Construction.
1st	Plain concrete.
2d	With $\frac{1}{4}$-in. bar at the bottom.
3d	" $\frac{1}{2}$-in. " " "
4th	" $\frac{3}{4}$-in. " " "
5th	" $\frac{7}{8}$-in. " " "
6th	" 1 -in. " " "
7th	" $1\frac{1}{4}$-in. " " "
8th	" two 1 -in. bars at the bottom.
9th	" two $1\frac{1}{4}$-in. " " "
10th	" two $1\frac{1}{4}$-in. " " " and one $\frac{1}{2}$-in. bar at the top.
11th	" two $1\frac{1}{4}$-in. " " " " two $\frac{3}{4}$-in. bars " "
12th, 13th, 14th	These were like Beams 9, 10 and 11, respectively, except that, in addition, fourteen U's of $\frac{1}{4}$-in. steel were imbedded in the beams, passing under the bars, and the ends projecting above the top of each beam.

All bars were put in straight, with the center 2 ins. from the top or bottom face of the beam. Many interesting facts were learned in these tests, but only such as relate to the subject in hand will be mentioned.

The component parts of the specimens were tested. The concrete in all cases consisted of one part by measure of Portland cement, three

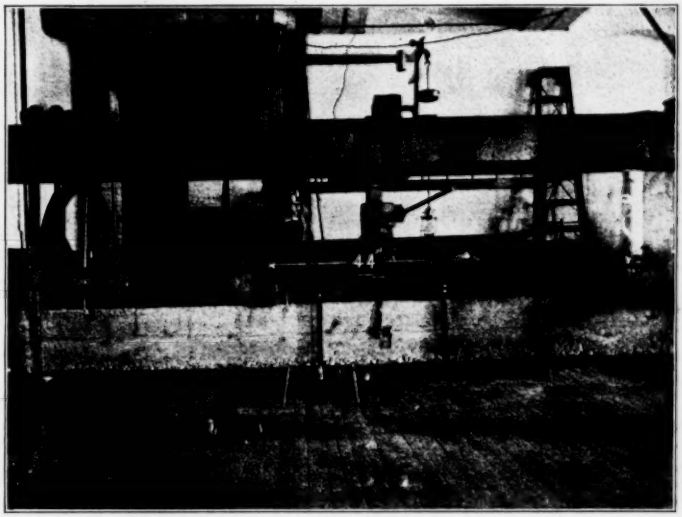

FIG. 1.—TESTING STEEL-CONCRETE BEAMS.

FIG. 2.—TESTING STEEL-CONCRETE BEAMS.

parts of Plum Island beach sand, and six parts of broken stone. The sand was perfectly clean; practically all passed through a 20-mesh sieve and was retained on a 30-mesh sieve; the grains, however, were round and smooth. The stone was a dark-blue trap; the pieces were cubical and the fractures angular. Four parts passed a $1\frac{1}{4}$-in. ring, two parts a $\frac{3}{4}$-in. ring. The steel was of a mild quality, square section, and was twisted cold until the edges made an angle of 20° with the axis of the bar. The cement was tested neat, and as 1 to 3 mortar. It developed nearly full strength in two weeks, so that there was but little increase in strength between 15 and 60 days. In the neat cement test, age 21 days, the average of five tension briquettes was 924 lbs.; highest, 978 lbs.; in compression, age 26 days, average of four, 8 140 lbs. per square inch; highest, 10 950 lbs. Mortar tension, age 12 days, average of four, 265 lbs.; the highest, 288 lbs. Mortar taken from concrete mixed by workmen, age 28 days, average of seventeen, 307 lbs.; highest, 368 lbs. Mortar by workmen, in compression, age 28 days, average of seven, 3 630 lbs.; highest, 4 500 lbs. Mr. Wason.

A concrete prism $8 \times 8 \times 12$ ins. was also tested, and developed a strength of 2 455 lbs. per square inch in compression. A $3\frac{5}{8}$-in. cube of stone failed at 22 000 lbs. per square inch. Some cubes of slightly larger size resisted the full power of the testing machine, and successfully withstood repeated stress of 20 000 lbs., both with and across the grain.

The average ultimate strength in tension on the steel bars of all sizes used was 83 330 lbs. per square inch, the highest being the $\frac{1}{2}$-in. bars which carried 100 140 lbs. The average modulus of elasticity was 30 095 000. The elastic limit averaged $64\frac{1}{2}\%$ of the ultimate strength.

The average weight of the concrete in the beams was 148.7 lbs. per cubic foot. The series was so made that the first specimen was quite weak in tension and the others were reinforced with steel until there was an excess of strength on the tension side, when the compressive portion was reinforced with steel to assist in its resistance to compression. The beams were all made in wooden forms inside the building, and left undisturbed until the time of these tests, which varied from 30 to 50 days.

The method used (Fig. 1, Plate VIII), was to rest the ends on small iron plates on the knife-edges of the testing machine, with a clear span of 11 ft. Blocks were laid on top of the specimen, dividing the span into three equal parts, and an I-beam was placed on top of these and the load applied through it. Thus a uniform bending moment was obtained between the two blocks. Yokes, 24 ins. apart, were attached rigidly to the middle portion of the beam, and micrometer readings were taken on the top and bottom as the beam was loaded. Assuming the plane of the yokes A B to be a plane, after the load was

Mr. Wason. applied, $A'\,B'$, we can find the position of the neutral axis as follows (see Fig. 1):

By similar triangles:

$$H : h :: x + y : y.$$

Therefore, $h = \dfrac{Hy}{x + y}$; H being the height of the beam, h the height of the neutral axis from the bottom of the beam, x the compression on the top, and y the elongation at the bottom. Clamps were attached to the beam over the supports, and fine piano wire was hung over these on either side of the beam, and, by means of micrometers attached to each side of the beam at the center, the deflection could be determined.

The plain concrete beam failed under a concentrated load of 2 500 lbs. at the center. Adding to this half the weight of the beam, namely, 600 lbs., and solving by the usual beam formula, the outside fiber stress is found to be 533 lbs. per square inch. The beam containing the ¼-in. bar failed at the same load. The bar pulled in two. If it is assumed that the bar was stressed to its limit, the concrete still had to carry 381 lbs. to the square inch in tension, as a concrete beam. The beam containing the ½-in. bar carried a greater load than was possible if the bar was required to carry the entire tensional stress. The bar pulled in two. If it is assumed that the bar was loaded to its ultimate strength, the concrete, as a beam, must still have supported part of the load, and the outside fiber stress would be 303 lbs. per square inch. Similarly, in the case of the ¾-in. bar, the concrete must carry 462 lbs. per square inch in tension. The ⅞-in. and 1-in. bars were capable of carrying the entire load coming on the beam containing them. The beam containing the 1¼-in. bar was accidentally cracked in two vertically in the middle in handling it before putting it into the machine, and carried 3 000 lbs. less concentrated at the center than the beam containing the 1-in. bar. As the 1-in. bar had an excess of strength on the tension side, there was no gain by the use of a 1¼-in. bar, and it is, therefore, safe to consider that the falling off in strength was due to the cracking of the concrete. On this assumption, the concrete in the beam with the 1-in. bar carried a stress of 516 lbs. per square inch. In the same way, the beam containing two 1¼-in. bars can be compared with the beam containing two 1-in. bars, which was accidentally cracked in two vertically in the middle; the ultimate strength of the former being 23 000 lbs. concentrated at the center, and of the latter 20 000 lbs. The 3 000 lbs. concentrated at the center would have to be carried by the concrete as a beam, and this would produce an outside fiber stress in tension of 516 lbs. per

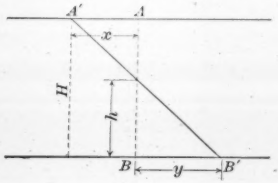

FIG. 1.

square inch, as in the previous case. The average of the foregoing results of maximum outside fiber stress in tension would be 466 lbs. per square inch. Mr. Wason.

The neutral axis was determined for each beam, and the total compression on the area above the neutral axis was calculated. This, in some cases, was easily done. With the $\frac{3}{4}$-in. bar, age 38 days, the maximum outside compressive stress was 2 916 lbs. per square inch; $\frac{7}{8}$-in. bar, age 50 days, 3 161 lbs.; 1-in. bar, age 50 days, 2 544 lbs.; $1\frac{1}{4}$-in. bar, age 41 days, 2 338 lbs.; and two $1\frac{1}{4}$-in. bars, age 42 days, 2 975 lbs.; the average of these being 2 787 lbs. per square inch. The beam containing two 1-in. bars (Fig. 2, Plate VIII), age 41 days, was cracked in two in the middle from top to bottom in handling, therefore the entire tension must have been carried by the steel and the entire compression was in the concrete. The neutral axis in this case was 10.2 ins. from the bottom, and this gave a maximum outside fiber stress of 8 588 lbs., which is very exceptional; yet, on account of being cracked in two, the stresses were more easily analyzed, and the result is correct.

In no case did a beam crack before the steel had reached its elastic limit. The average deflection in 11 ft. for twelve beams was 0.47 in., and the maximum was 0.76 in. when the first crack appeared. Every fracture was free from air holes, and showed dense, evenly-mixed concrete. In many cases the stone sheared in two without pulling from the mortar.

The formula used by the speaker for calculating concrete beams is:

$$f = \frac{W l}{6\frac{2}{3} D};$$

where W is the total load in pounds, l the span in inches, D the depth in inches from the top to the center of the bar, and f the total fiber stress in the steel. This formula was deduced at least twelve years ago. The proof follows:

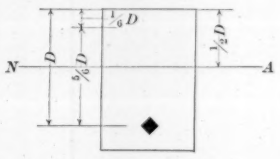

FIG. 2.

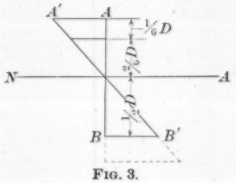

FIG. 3.

Assume as the fundamental condition an absolutely perfect bond of union between the steel and the concrete within the limits of the working strength of the combination. Assume that the steel takes the entire tensile stress, and the concrete the entire compressive stress. The concrete below the bar is for the purpose of holding it in place. While the concrete below the neutral axis certainly is in tension, it is

Mr. Wason. not considered in the computation. Assume the neutral axis half way between the center of the bar and the top of the beam. The center of pressure of the compressive area is two-thirds of the height from the neutral axis to the top, and the distance between this point and the center of the bar is five-sixths of the depth. This is the arm of the moment of internal forces, which is $\frac{5}{6}Df$. For a uniformly distributed load the moment of the external force would be $\frac{Wl}{8}$. Equating the two moments and solving for f, we get the formula,

$$f = \frac{Wl}{6\frac{2}{3}D}.$$

After solving and finding the total stress on the bar, a size and spacing can be selected which is convenient, and the area of concrete can be adjusted to resist the compression. The unit which the speaker has used, 16 000 lbs. per square inch in tension on steel, and for a mixture of concrete, 1:3:6, an average of 500 lbs. per square inch in compression on concrete, requires 32 sq. ins. of concrete in the upper third of the beam for each square inch of steel.

A good many pieces of actual work have been loaded with three times their calculated amount, and have shown no sign of failure whatsoever; therefore, structures calculated in this way are safe. The beams tested at the Massachusetts Institute of Technology carried from 4½ to 5 times the load they were calculated to carry, according to the foregoing formula. In these tests, with all bars 2 ins. from the bottom, the neutral axis would be 7 ins. from the bottom. As a result of measurements on twelve beams, although there was considerable variation in the position of the neutral axis, the highest being 10.2 ins., the lowest 4.74 ins., the average was 7.06 ins., or sufficiently near to check the assumption made in the formula. This is an extremely simple formula for every-day use, and while it may not conform to profound theory, it certainly gives safe results in the design of steel-concrete structures. The speaker believes this formula can be used for any composite construction within the safe limit of the bond of union between the steel and the concrete.

Some experiments on the bond of twisted steel have been made, as follows: ½-in. bars were imbedded in blocks of concrete 6 ins. square and which varied in length from 12 to 28 ins. The bars projected about 2 ft. from one end of the blocks. One face of the concrete bore against a plate, the bar passing through a hole in its center. A direct pull was applied to the bar, and in each case, when the length exceeded 21 ins., the bar broke outside the concrete without pulling therefrom. In these cases the ultimate stress on the steel averaged 81 835 lbs. per square inch. From the data thus obtained it is possible to determine the exact length for bars of other sizes, and when this limit is exceeded there is no danger whatever of the bars drawing,

and therefore the formula can be applied. If 16 000 lbs. is the working stress on the bar, it is only necessary to imbed it one-fifth of the distance as just determined in order to get the safe working bond required by the formula. Mr. Wason

This formula can be applied to proportion the section of steel in a bridge. After determining the bending moment in the usual way, insert this in the formula, and use for the depth the distance from the center of the bar to the other face of the arch; then solve, and the size of the bar which will resist this moment is found. If other stresses, such as temperature, are reduced to the proper form, the steel can be proportioned in the same way to resist these stresses.

The neutral axis for each beam was computed from theoretical grounds, as follows:

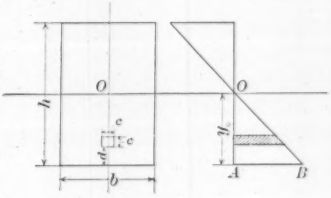

FIG. 4.

Let h = height of beam;
y_o = height of neutral axis from the bottom;
b = breadth of beam;
c = breadth of square rod;
d = height of rod from bottom of beam;
a = strain at one unit distance from origin;

$$ay_o = AB = \frac{f}{E};\ a = \frac{f}{Ey_o}.$$

At distance y from O, $a_1 = \frac{f}{Ey_o}y$. Stress in concrete $= Ea_1$; in steel $= E_1a_1$.

Beam with one rod in the bottom:

$$E\int_{-\frac{b}{2}}^{\frac{b}{2}}\int_{o}^{y_o} a_1\,dx\,dy - E\int_{-\frac{b}{2}}^{\frac{b}{2}}\int_{o}^{h-y_o} a_1\,dx\,dy$$

$$-E\int_{-\frac{c}{2}}^{\frac{c}{2}}\int_{y_o-c-d}^{y_o-d} a_1\,dx\,dy + E_1\int_{-\frac{c}{2}}^{\frac{c}{2}}\int_{y_o-c-d}^{y_o-d} a_1\,dx\,dy = 0.$$

$$Eb\int_{o}^{y_o} a_1\,dy - Eb\int_{o}^{h-y_o} a_1\,dy - Ec\int_{y_o-c-d}^{y_o-d} a_1\,dy + E_1c\int_{y_o-c-d}^{y_o-d} a_1\,dy = 0.$$

$$Eb\left[\frac{y_o^2}{2} - \frac{(h-y_o)^2}{2}\right] - Ec\left[\frac{(y_o-d)^2}{2} - \frac{(y_o-c-d)^2}{2}\right]$$

$$+ E_1c\left[\frac{(y_o-d)^2}{2} - \frac{(y_o-c-d)^2}{2}\right] = 0.$$

$$\frac{Eb}{2}(2hy_o - h^2) + \frac{(E_1-E)c}{2}[d^2 - (c+d)^2 + 2y_oc] = 0.$$

Mr. Wason.

Two rods in the bottom:

$$\frac{E\,b}{2}\,(2\,h\,y_o - h^2) + (E_1 - E)c\,[d^2 - (c+d)^2 + 2\,y_o\,c] = 0.$$

Assuming the modulus of elasticity for concrete to be 1 000 000 and for steel to be 28 000 000, the calculated neutral axis was lower in every case than the observed, the average height being 4.77 ins. from the bottom. Assuming the modulus to be 2 500 000, the average height of the neutral axis, as calculated by the foregoing formula, was 5.37 ins. from the bottom. These results differ too much from the observed to be reconciled with one another, and therefore the assumptions made in the deduction of the formula do not agree with practice.

Curves were plotted for each beam, using the deflections as ordinates and the bending moments as abscissas, and in every case a curve was obtained. The curve, however, as a rule, was quite flat. Nevertheless, the fact that it was a curve proves that Hooke's Law does not apply exactly to a composite beam of this type.

Five years ago the speaker had a contract, in the Fall River Public Library, in which it was necessary to bring a load of 150 000 lbs. upon a pier of concrete 15 ins. square, and not being able to find any printed data of tests on large specimens, he made a series of experiments* at the Watertown Arsenal on piers 1 ft. square and varying in length from 2 to 14 ft. There were three series. In the first he used all hand-mixed concrete, 1 : 3 : 6, of the proportion given previously for beams, and of practically the same quality. The second series was exactly similar, except that the concrete was machine-mixed; the third series was of leaner mixtures.

The specimens were all mixed in the open air in midwinter, and filled into vertical moulds, in an open building without heat, and were allowed to freeze and thaw, according to the weather, until tested, at an age of from 35 to 47 days. It is the opinion of the speaker, from observation and research, that had these specimens been made in midsummer and tested at the same age, they would have developed about twice the ultimate strength recorded herein. The longest specimens were tested first, and were put into the Emery machine in a horizontal position. The counterweight hung from the center did not balance the weight of the specimens themselves, so that there was a bending moment producing an outside fiber stress of 40 lbs. per square inch. Nevertheless, every specimen failed by direct compression, the longest ones showing the greatest strength, although they did not have as long a time to set as the short specimens. The average of the hand-mixed series was 989 lbs. per square inch; of the

* For full report of these tests, see the Report of the Watertown Arsenal for the year 1897, "Tests of Metals and Other Materials for Industrial Purposes," pp. 349–385.

machine-mixed, 1 098 lbs. per square inch, an advantage of 11% in favor of machine-mixing. Mr. Wason.

Readings of the compression were taken on all beams 6 ft. and more in length, some with repeated stress, some with a long lapse of time between stresses, and many interesting facts were developed. For instance, a load of 800 lbs. was put on the 12-ft. column, No. 8763, and the compression measured immediately. After a lapse of 5 minutes the beam had regained a small portion of its compression. The same was true of the 14-ft. column, No. 8759. With the 10-ft. column, No. 8766, exactly the reverse took place. The initial load in every case was 100 lbs. per square inch, and was increased by 100-lb. increments until the elastic limit was reached. The instruments were then removed and the specimens broken. After each application of load, it was removed, and the specimens regained their entire compression up to 500 lbs. per square inch in most cases, and above this point regained over two-thirds of the amount they were compressed, showing the material to be quite resilient.

Curves were plotted with the compressions as ordinates and the loads per square inch as abscissas, and these were quite flat, but still sufficiently curved to show that Hooke's Law does not apply closely, except within the limit of 700 lbs. per square inch. At this load the modulus of elasticity for hand-mixed concrete was 2 500 000 for an average of ten tests; on machine-mixed concrete it was 2 870 000 for an average of ten tests. The highest result of all obtained was 4 545 000. The third series, all machine-mixed, was made with specimens all 6 ft. in length, the first having a mixture of 1 cement, 3 of sand and 6 of stone, the crushing strength being 1 166 lbs. per square inch; second, 1:4:8 mixture, crushing strength 849 lbs. per square inch; third, 1:5:10, crushing strength 728 lbs. per square inch; fourth, 1:6:12, crushing strength 483 lbs. per square inch; fifth, 1:7:13, crushing strength 432 lbs. per square inch. A curve was plotted for these, using for ordinates a ratio of the volume of cement to the volume of concrete, and for abscissas pounds per square inch, and it was found that the points could be represented by a straight line, thus showing that, within these limits, the strength of the concrete is directly proportional to the amount of cement used.

Another test of the relative strength of hand and machine-mixed concrete might be mentioned. A contractor got into an argument with the supervising engineer on a piece of work over the quality of concrete mixed by a certain type of concrete mixer. This was settled by a test of 1-ft. cubes made at the Watertown Arsenal. The concrete was composed of 1 part of Giant cement, 3 parts of sand direct from the bank, and 5 parts of broken stone passing a $2\frac{1}{2}$-in. ring, with all material passing a $\frac{1}{2}$-in. ring screened out. The specimens were tested at 90 days. The machine-mixed specimens averaged 4 263 lbs. per

Mr. Wason. square inch; the hand-mixed, which were a little better in quality than is obtained in every-day practice, averaged 3 187 lbs. per square inch; an advantage of 25¼% in favor of machine-mixing. The machine in question was the Portable Gravity Concrete Mixer.

The speaker also made a groined vault which was as near as possible an eighth-scale model of a room 27 ft. 9 ins. by 33 ft. 6 ins., the specimen being 52 ins. long and 42 ins. wide. Arch ribs sprung diagonally from one corner to the other with a rise of one-tenth of the span. The arch vault sprung between them with a rise equal to that of the ribs at the center. The ribs were 1 in. wide and 2 ins. deep, the vaults ½ in. thick. The concrete was composed of 1 part cement, 2 parts sand and 4 parts stone grit. A load was applied at eight points near the crown, four on the ribs and four on the vault, and the specimen carried 26 605 lbs. before failing.

All the foregoing figures would indicate that a somewhat higher working stress can be used in concrete than has been customary. The speaker, in his practice, has been in the habit of using for a 1:3:6 concrete (his standard), 500 lbs. per square inch in compression, 80 lbs. per square inch in tension, and a ratio of steel area to concrete area of 1 to 32, using 16 000 lbs. as the stress in steel. While the foregoing experiments would indicate that still higher pressures are possible with safety, the speaker does not recommend any increase until these results have been checked by other experimenters. With the rapid increase in the quality and uniformity of Portland cement, he believes that in a short time it will be possible to use larger unit stresses than are now customary; or leaner mixtures, with the present unit stresses, without any danger whatever of exceeding safe limits of strength.

Mr. Johnson. A. L. JOHNSON, Assoc. M. Am. Soc. C. E.—While steel and concrete have been used in combination for hundreds of years, the real scientific genesis of steel-concrete construction is comparatively recent, and much experimental work yet remains to be done before the subject is fully understood. This work is being rapidly prosecuted, however, especially abroad, and it will probably not be long before the science is fully developed. We will then have our steel-concrete engineers as we now have our bridge engineers.

So far as the speaker has been able to discover, the first man to use these materials in a scientific manner, the metal being imbedded in the lower, or tension, side of the concrete, was W. E. Ward, who in 1875 constructed a building at Port Chester, N. Y.,* in which "not only all the external and internal walls, cornices and towers were constructed of béton" (the word concrete was not then in use), "but all the beams and roofs were exclusively made of béton, reinforced with light iron beams and rods."

In 1869 François Coignet, of Paris, took out letters patent on a

* *Transactions*, Am. Soc. Mech. Engrs., Vol. iv, p. 388.

combination of béton and iron rods, but he had no conception of the proper methods of using the materials. Neither had Monier, who, in 1876, built some flower-pots of béton and wire. He used the iron merely as a webbing, which was preferably placed in the neutral axis of the slab. For circular tanks or such constructions as placed the whole cross-section in tension this was not an improper method, but was, of course, of no value in resisting a bending stress. Mr. Johnson.

Ward, however, had the real scientific idea, and built rods into the lower side of his concrete beams and floor joists much as is done to-day. He had no formulas to guide him, but relied upon his judgment.

In the following year, 1876, Thaddeus Hyatt, of England (recently deceased), began some experimental work on concrete and metal construction, his tests being made by Kirkaldy, and in 1877 he published a book giving the results of his experiments. The tests were interesting and of much value in the development of the science.

Perhaps the first approximately correct formulas for the strength of this class of construction were derived by Julius Mandl, in Germany, by the method of least work, and about the same time, in this country, by J. B. Johnson, M. Am. Soc. C. E., using another method, both giving the same equations.

Later, in 1898, L. A. Sanders, Engineer for the Monier construction at Amsterdam, published an elaborate treatise on the science of this construction.

All the foregoing investigators, however, took for the ultimate strength of the construction the proportionate elongation at which the concrete on the tension side would fail when no metal reinforcement was used. Given a certain modulus of elasticity for the concrete in tension, and a certain tensile strength, it seemed evident that this would fix the maximum proportionate distortion obtainable in the concrete. Then when the concrete had failed on the tension side it was assumed that the formulas would no longer be available, and that, even if the maximum strength had not been reached, the construction was in an unsafe condition owing to the exposure of the metal reinforcement.

The speaker believes he was the first to point out, at least in this country, the error of these assumptions, and that a section plane before bending was still a plane after the concrete on the tension side had passed far beyond its limit of strength. He had observed on tests made under his direction that the cracks which certainly ought to have been there were not visible even on a plastered surface with five to ten times the maximum proportionate distortion obtainable when no metal reinforcement was used.

It was his idea at this time that the metal served as an equalizer of slight variations in the strength at different places in the concrete,

Mr. Johnson. and that the cracks, though undoubtedly present, were so greatly distributed and divided up as to be too fine to be visible. This being the case, it seemed evident that for all practical purposes, at any rate, a section plane before the point was reached at which this rupture was supposed to occur would still be a plane after passing this point; in which case the same formulas would apply, it being only necessary to omit from the moment of resistance the moment due to the concrete in tension, which did not amount to very much anyway.

The next important contribution to the literature on this subject seems to have been made by M. Considère, Ingénieur en Chef des Ponts et Chaussées, Paris, who published* quite a long discussion on the subject, the basis of which was a series of tests made by him on specimens about 24 ins. long by 2½ ins. square, subject to uniform bending moment throughout their middle portion. The specimens were made of a Portland cement and sand mortar. some having metal imbedded and others of the same batch with no metal. The metal was round iron rods or wire, no transverse metal being used, to avoid complications in the stresses.

His studies embrace the following points: Graphical and algebraic determinations of the moment of resistance of armed pieces, influence of the quality of the concrete and of the armatures, the most economical percentage of metal, effect of bad workmanship, value of the factor of safety, and the utility of symmetrical armatures, by which is meant those running in two directions at right angles to each other such as is used in Monier constructions. M. Considère's article marks a distinct advance in the science. While it is impossible here to enter into a description of the experiments and a discussion of the paper—in fact, it contains much that is neither new nor of especial interest in this country—it is necessary to mention certain important results obtained, having as they do, an enormous influence on the scientific design of steel-concrete structures.

These results, as to which the evidence seems conclusive, are as follows:

1.—In armed concrete beams, the concrete on the tension side will submit, without rupture, to a proportionate distortion of from ten to twenty times that at which it would fail in an unarmed direct tension test. It will also have, during the additional period of distortion, a strength nearly equal to its maximum strength in direct tension.

2.—Severe interior stresses are introduced in armed concrete constructions where rich concrete mixtures are used, owing to the shrinkage of the latter during the first eighteen months if exposed to the air, or a corresponding swelling during the period if in submarine work. For neat cement this amounts to 2 mm. per meter, decreasing with the quantity of base material added.

* *Le Génie Civil*, February 4th to 25th, 1899.

3.—These interior stresses are, to a considerable extent, relieved by the slipping of the bars or rods in the concrete, as to which action he says his tests leave no room for doubt. The stress is also relieved in time, if cracks are not previously developed, by what he calls the tendency of the concrete to eventually yield somewhat to a soliciting force. Mr. Johnson.

With regard to the determination of the first result, M. Considère subjected an armed beam to a bending giving 20 times the proportionate distortion in the concrete in tension at which it should have failed, and then subjected it to 139 repetitions of loadings where the proportionate distortion in the concrete ranged from $4\frac{1}{2}$ to 13 times the maximum obtainable from an unarmed piece. He then carefully cut out an annular ring of concrete surrounding one of the rods and found not only that it was not cracked, but that it was still capable of carrying nearly its maximum load.

It seems scarcely possible to obtain more conclusive evidence than this. It is certainly a great surprise to the speaker, who has always believed that the cracks were really there, though so fine as to be invisible.

The next result was determined from a neat-cement specimen, $1 \times 2\frac{1}{2}$ ins. in cross-section, having a $\frac{4}{10}$-in. rod imbedded in the center. The rod was originally measured, imbedded in the concrete, and then after being immersed in water for some time, measured again. The cement was then stripped off and the length again determined, when it was found to be of the same length as originally, though having been materially longer on the second measurement. In this way it was ascertained that the rod had been subjected to a tensile stress of 17 070 lbs. per square inch, and the concrete to a compressive stress of 937 lbs. per square inch.

These values are very high, and render the advisability of the use of very rich mixtures in steel-concrete construction questionable.

The third result obtained, however, indicates that relief from this stress will probable be afforded on account of the slipping of the bars in the concrete, and the tendency of the concrete to gradually yield to a continuing force. The speaker fails to see any consolation whatever in this.

Accepting the reliability of these three results, and summing up their effect upon the design of steel-concrete structures, we arrive at the following conclusions:

1.—The maximum moment of resistance of steel-concrete beams should include the tensile strength of the concrete as one of the elements thereof, operating to almost its maximum capacity. There will be no cracks in the concrete on the tension side previous to the development of this maximum moment.

2.—On account of excessive shrinkage stresses, we should avoid, in

Mr. Johnson. steel-concrete construction, the use of very rich mixtures of concrete. The speaker would construe this as referring more particularly to the usual mortar mixtures rather than to the ordinary proportions in concrete work.

3.—To avoid the slipping of the metal in the concrete, dependence should not be placed solely upon the adhesion of the concrete to the surface of the metal; but the rods should be so constructed as to have an additional and more reliable form of bond.

The adhesion is very great originally, and entirely sufficient for our needs; but the slipping undoubtedly occurs through the weakening of this adhesion by vibrations and shocks to which, more or less, all structures are subjected.

It is entirely apart from the purpose of this discussion to present anything as to the characteristics or relative merits of the various styles of steel-concrete construction now on the market, many of which are protected to a greater or less degree. The foregoing conclusions, however, give reliable criteria for the determination of these merits.

That the concrete does not fail at all in tension, for proportionate distortions amounting to 20 times as much as it ought to be able to stand, is to the speaker the most interesting of M. Considère's results. As to the others, they only serve to substantiate previous impressions.

It is certain that the cracks do not show, but that they do not exist would be impossible of acceptance except on the strength of such elaborate and precise experiments as M. Considére has conducted.

To the speaker's knowledge, nobody as yet has confirmed the first result, and it may be that many engineers will wait for this before accepting it. For this reason the formulas following are based upon the concrete in tension being of no value in the moment of resistance of the cross-section. We are all the more willing to make this concession to the natural conservatism of careful engineers, since, after all, the effect is inconsiderable except where very rich mixtures are used, which, as has been shown, are not advisable in this class of construction.

Let E_s = Modulus of elasticity of steel, in pounds per square inch;

E_c = Modulus of elasticity of the concrete in compression, in pounds per square inch;

F = Elastic limit of steel, in pounds per square inch;

f_c = Compressive strength of concrete, in pounds per square inch;

b = Width of section considered, in inches;

a^2 = Area of bars, in square inches;

d = Spacing of bars, in inches;

$\frac{a^2}{d}$ = Amount of metal per inch in width, in square inches;

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M_o = Moment of ultimate resistance of cross-section, in inch-pounds, for failure in tension;
M_o^1 = Moment of ultimate resistance of cross-section, in inch-pounds, for failure in compression;
M = Bending moment of external forces, in inch-pounds;
W = Total load on beam, in pounds;
w = Uniformly distributed load, in pounds per square foot;
P_s = Total stress on metal in width b;
P_c = Total stress on concrete in width b;
λ_1 = Unit elongation of extreme fiber in compression;
λ_2 = Unit elongation of steel;
e = The distance from the bottom of the concrete to the middle plane of the metal, in inches.

As we cannot count on developing, at the maximum load, a stress in the metal much greater than its elastic limit, it is well to use steel with as high an elastic limit as is consistent with such ductility as may be necessary in the particular construction used. Though it is often possible in these constructions to break the metal reinforcement entirely in two, yet the high stress in the metal which this implies is not developed until long after the construction as a whole has attained its maximum carrying capacity.

It will be assumed, therefore, that the modulus of elasticity of the steel is constant up to the development of the value F, which is practically the elastic limit, at which time the construction as a whole has reached its maximum resistance.

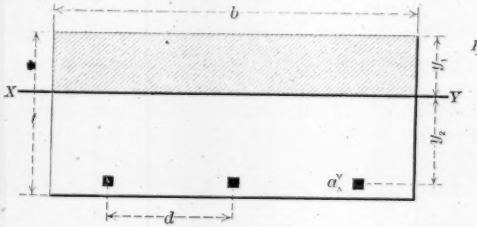


FIG. 5.

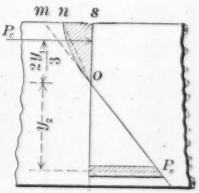


FIG. 6.

This condition does not hold on the concrete side. Here, the maximum load-carrying capacity will occur when the extreme fiber stress on the concrete becomes equal to its compressive strength, assuming that there is enough steel used to develop it. When this occurs, the stress diagram will be as shown above the neutral axis in Fig. 6. On this diagram the total stress P_c, equal to the area of the shaded portion, is about 25% greater than the area between the lines $o\,n$ and $o\,s$. The speaker has examined a large number of stress diagrams and finds that this is true within reasonably close limits for both rock and cinder con-

Mr. Johnson.

cretes of the usual mixtures. Also, in the same way, it was determined that the modulus of elasticity as represented by the tangent of the angle $n\ o\ s$ is in rock concrete only two-thirds and in cinder concrete one-half as much as the value of the original modulus, represented by the tangent of the angle $m\ o\ s$, which is the one usually given in test reports.

Single-Span Beams on Knife-Edge Supports.—Referring to Figs. 5 and 6 we have

$$P_s = \frac{F a^2 b}{d};$$

Then the strain, or stretch of the metal is,

$$\lambda_2 = \frac{F}{E_s},$$

$$\text{and } P_s = E_s \lambda_2 \frac{a^2 b}{d}.$$

On the compression side we have,

$$\lambda_1 = \lambda_2 \frac{y_1}{y_2}, \text{ and the resulting stress is,}$$

$$f_c = \lambda_1 \frac{2 E_c}{3} \text{ for rock concrete, and } \lambda_1 \frac{E_c}{2} \text{ for cinder concrete;}$$

$$= \lambda_2 \frac{y_1}{y_2} \frac{2 E_c}{3} \text{ for rock concrete, and } \lambda_2 \frac{y_1}{y_2} \frac{E_c}{2} \text{ for cinder concrete.}$$

$$\text{Also } P_c = \frac{5 f_c b y_1}{8} \text{ for both rock and cinder concretes.}$$

$$\text{Then } P_c = \frac{5 b y_1^2 \lambda_2 E_c}{12 y_2} \text{ for rock concrete,}$$

$$\text{and } = \frac{5 b y_1^2 \lambda_2 E_c}{16 y_2} \text{ for cinder concrete.}$$

$$\text{But } P_c = P_s; \text{ hence,}$$

$$E_s \lambda_2 \frac{a^2 b}{d} = \frac{5 b y_1^2 \lambda_2 E_c}{12 y_2} \text{ for rock concrete,}$$

$$\text{from which } y_1^2 = \frac{12 a^2 y_2 E_s}{5 d E_c} \text{ for rock concrete.} \quad (1)$$

$$\text{Also } E_s \lambda_2 \frac{a^2 b}{d} = \frac{5 b y_1^2 \lambda_2 E_c}{16 y_2} \text{ for cinder concrete,}$$

$$\text{from which } y_1^2 = \frac{16 a^2 y_2 E_s}{5 d E_c} \text{ for cinder concrete} \quad (2)$$

For the moments of resistance of the cross-section we make no appreciable error if we assume that the location of the resultant pressure on the compression side, or the center of gravity of the upper shaded area in Fig. 6, is where it would be if the figure were a triangle.

Mr. Johnson.

We then have,

$$M_o = P_s\, y_2 + \frac{2\, P_c\, y_1}{3} = \frac{P_s}{3}\,(3\, y_2 + 2\, y_1) = \frac{F\, a^2\, b}{3\, d}\,(3\, y_2 + 2\, y_1) \ldots (3)$$

$$M_o^1 = P_s\, y_2 + \frac{2\, P_c\, y_1}{3} = \frac{P_c}{3}\,(3\, y_2 + 2\, y_1) = \frac{5\, b\, f_c\, y_1}{24}\,(3\, y_2 + 2\, y_1) \ldots (4)$$

We also have,

$$t = y_1 + y_2 + e \ldots (5)$$

The value e varies with the thickness and also the character of the metal. It should not be less than $\frac{1}{2}$ in. for bars, nor less than the thickness of the bar used. In such a material as expanded metal, in which the metal runs along planes inclined with the horizontal, this distance may be as small as $\frac{1}{4}$ in. In calculating any given construction the value e is first inserted in (5) which is then combined with either (1) or (2), depending upon which kind of concrete is used, to get the values of y_1 and y_2. These values inserted in (3) and (4) give the moments of resistance, the smaller of the two values being used as the strength of the section.

Single-Span Beams on Knife-Edge Supports of Equal Strength above and below the Neutral Axis.—Here M_o becomes equal to $M^1{}_o$ or,

$$\frac{F\, a^2\, b}{3\, d} = \frac{5\, b\, f_c\, y_1}{24}, \text{ from which } \frac{a^2}{d} = \frac{f_c\, y_1}{1.6\, F} \ldots (6)$$

Combining (1) and (6), and (2) and (6) we have,

$$y_2 = \frac{2\, F\, E_c\, y_1}{3\, f_c\, E_s} \text{ for rock concrete} \ldots (7)$$

and

$$= \frac{F\, E_c\, y_1}{2\, f_c\, E_s} \text{ for cinder concrete} \ldots (8)$$

Introducing these values in (4), we have,

$$M_o^1 = M_o = \left[\frac{5\, b\, F\, E_c}{12\, E_s} + \frac{5\, b\, f_c}{12}\right] y_1{}^2 \text{ for rock concrete} \ldots (9)$$

and

$$M_o^1 = M_o = \left[\frac{5\, b\, F\, E_c}{16\, E_s} + \frac{5\, b\, f_c}{12}\right] y_1{}^2 \text{ for cinder concrete} \ldots (10)$$

In designing, where one is using a metal the quantity of which can be altered at will, then the construction should be made of equal strength in tension and compression, or $M_o = M_o^1$. In this case, if we desire to obtain a moment of resistance equal to a given bending moment, M, we have,

$M_o^1 = M$, and from (9) and (10)

$$y_1 = \pm \sqrt{\frac{M}{\frac{5\, b\, F\, E_c}{12\, E_s} + \frac{5\, b\, f_c}{12}}} \text{ for rock concrete} \ldots (11)$$

$$\text{and } y_1 = \pm \sqrt{\frac{M}{\frac{5\, b\, F\, E_c}{16\, E_s} + \frac{5\, b\, f_c}{12}}} \text{ for cinder concrete} \ldots (12)$$

Mr. Johnson. Equations (11) and (12) are very simple, as it will be noticed that, knowing the kind of concrete to be used, the radicals reduce at once to numerical fractions. When y_1 is thus determined we get $\frac{a^2}{d}$ from (6) and y_2 from (7) or (8). $\frac{a^2 b}{d}$ gives the amount of metal required for width b. The thickness of the concrete and the spacing of the bars will then only depend upon the size of the bars used.

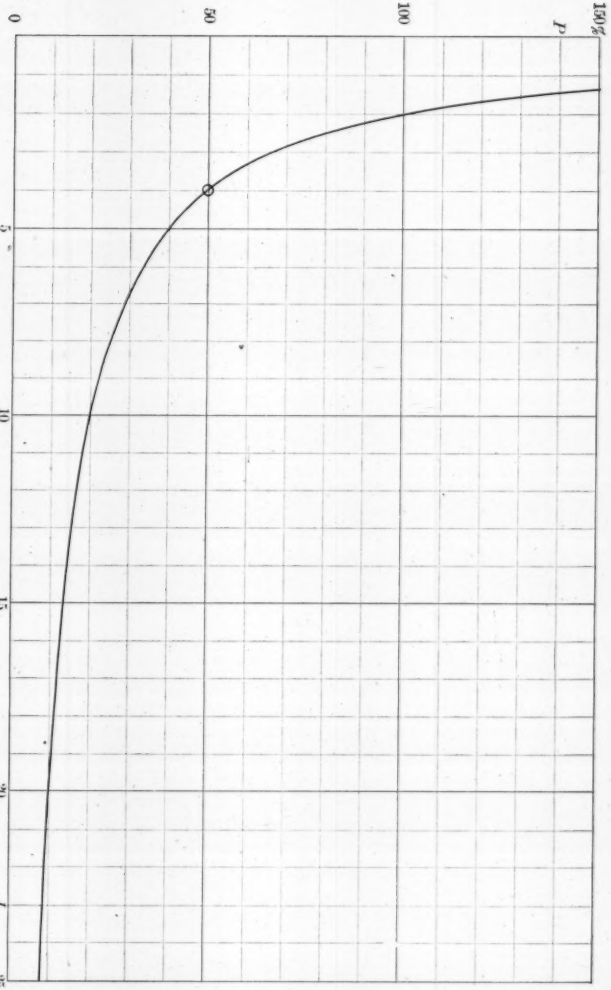
Floor Panels.—The foregoing formulas apply to steel-concrete, single-span beams, supported on knife-edges at the ends. The discussion is wholly inadequate when applied to the carrying capacity of such construction as is used in flooring where there are haunches on the beams and flooring in the adjacent panels. There has not, as yet, been developed a scientific discussion of the strength in such cases, and it is doubtful if there ever will be. It is known, in general, that a slab of a given thickness will carry more than twice as much, when incorporated in a floor, as it will when tested as a beam simply supported at the ends.

In fact, on spans of 4 ft. this ratio is more than 3 to 1, though probably this excess is entirely due to the fact that, in testing, a uniformly distributed load is generally used which will always arch considerably, no matter what the material may be. It is only fair, however, that the short spans should be allowed this advantage, as, when in use, the stored material cannot act otherwise. So far as concentrated loads in practice are concerned, except in special cases of rare occurrence, their effect is much over-estimated, and the special cases could receive individual treatment.

While, as has been said, it is, as yet, impossible to give a truly scientific discussion of these matters, we can arrive at an idea of the conditions by considering that the haunches on the beams produce a continuous-girder action, such that the external bending moment in the middle is only two-thirds as great as if it were free at the ends. Also that the floor in the adjacent panels produces an interior arching action within the thickness of the slab, adding about one-third to the area of the compressive stress diagram, this extra compression being counteracted by the adjacent floor panels instead of by the metal reinforcement. The effect of these two elements is to double the strength of the beams as used in floors.

On spans of 4 ft. in length uniformly loaded with any material not a grain or fluid, we can safely add another 50% to the moment of resistance on account of the arching effect previously alluded to. For zero span this effect would be infinite, and for infinite span it would be zero. We would then have, for the curve of correction to the moment of resistance, a hyperbola, as shown in Fig. 7, passing through a percentage of 50 at a span of 4 ft., and having the axes as asymptotes.

Mr. Johnson.



P = PERCENTAGE TO BE ADDED TO M_0 ON ACCOUNT OF THE ARCHING ACTION OF A UNIFORMLY DISTRIBUTED LOAD.
L = LENGTH OF SPAN IN FEET.

FIG. 7.

Mr. Johnson. Of course, the arching action varies with the deflection and also with the strength of the construction, the latter determining the height of the load. These are allowed for, in the speaker's experience with concrete and metal constructions, by the selection he has made of 50% at a span of 4 ft.

The whole argument as applied to floor panels is admittedly *a posteriori*, the speaker having had much experience with, at any rate, certain styles of construction, and knowing, therefore, in advance, about what his formulas should show. At least, the method has the merit of being safe.

The speaker's practice is, therefore, as follows:

For steel-concrete beams simply supported at the ends, use for the moment of resistance Formulas (3), (4), (9) and (10).

For floor construction, double the values given by the foregoing formulas and add to this a percentage depending upon the span length and obtained from Fig. 7.

For special cases of floor construction, where heavy concentrated loads are used, reduce the same to the equivalent non-arching uniformly-distributed load and then double the values in Formulas (3), (4), (9) and (10), and add no percentage.

The speaker has tables for the design of the special kinds of steel-concrete constructions used by him and for the concrete materials found in the territory of his operations.

As, however, these tables would not be applicable to other kinds of metal reinforcements, or to other localities where different kinds of materials are used in the concrete work, it is thought inadvisable to give them here.

It has been found that the elements of strength of concrete vary a great deal with different classes of materials, and the speaker would recommend that on any large work these functions be especially determined. This applies particularly to the compressive strength, and the compressive modulus of elasticity.

So far as floor construction is concerned, it is evident that the last word has by no means been said upon the subject of the theory of the combination of steel and concrete. But the speaker will be satisfied if he has shown that our knowledge is sufficient to give a safe and at the same time a fairly economical design.

As to simple beams, our knowledge is precise and sufficient.

So far as the other questions are concerned, with regard to the reliable use of these materials in combination, it would seem that they, too, have been practically all answered in the affirmative.

The coefficients of expansion are so nearly alike that no possible variations in temperature can produce any ill effects.

As to corrosion, bars may be imbedded in either rock or cinder concrete, if it is well mixed and has a reasonable quantity of cement, say

1:3:6, and left exposed to the elements for years and be as bright and clean when cut out as the day they were placed. Cinders containing a large percentage of sulphur will give just about as good a result, as these particles are covered over and rendered ineffective by the Portland cement water, or grout, that permeates the whole mass. Mr. Johnson.

As to its use in fire-proofing, where cinder concrete is used, the barriers have all been burned away, it having stood the severest test, both at home and abroad.

The admirable thing about this class of construction is the reliability with which a desired result can be obtained, and the performance again and again duplicated. One must, of course, know his materials. This is the quality that makes possible the development of the scientific theory, and renders it available for high-class engineering work.

The preceding part of this discussion applies particularly to flat-slab constructions rather than to deep beams. For such beams the value of the concrete in tension becomes of some consequence. It is impossible, however, to give a general expression for the moment of resistance, taking account of this element, without having an equation for the curve of the compressive stress diagram for the concrete used. Bach has derived such equations, exponential in form, and they have been used by L. A. Sanders, of Amsterdam, Engineer for the Monier Construction, in his pamphlet on "Cement-Iron Construction," published in 1899, of which the writer has made an English translation. But the equations were complicated and the treatment was incorrect in some particulars, especially with regard to fixing upon the proportionate distortion at which the concrete should fail in tension as the maximum capacity of the beam. This was known to be improper, even at that time, and later, M. Considère's results, previously referred to, gave precise information on this point.

Mr. Sanders handled his equations well, however, and demonstrated that it would be possible to take account of the curvature of the compressive stress diagram in the equations for the moment of resistance. To get even an approximately correct formula for the moment of resistance of a given construction made up haphazard, it would be necessary to introduce in the formula an equation expressing this relation between the stress and the strain, or f_c and E_c.

For the writer's part, he is not particularly interested in knowing how badly or how well other work has been designed, his chief concern being to try and design correctly himself; and for such cases the formulas can be simplified.

The correctly designed beam will be one of symmetrical strength; one equally likely to fail above or below the neutral axis. This means that the full compressive strength of the concrete will be developed, and that the full elastic limit of the steel will be reached.

When these two things occur it is known that the concrete in tension

Mr. Johnson. has long passed the time at which its maximum strength was arrived at, which strength, nevertheless, is still intact.*

We then have, with reference to the concrete in compression, the same relations as those assumed in the first part of the writer's discussion. That is to say, we are using the whole of the compressive stress diagram, and while these diagrams vary much as to the angle the tangent makes at the origin, or the initial modulus of elasticity, yet the general relation of the total area of this diagram to the inscribed triangle, and of the original modulus to the resultant modulus for the maximum stress and strain, will not be greatly altered. By taking constants for these relations, which of course only apply to the maximum values of stress and strain, we obviate the necessity of introducing a complicating expression for these relations, which would, whoever, hold good for all points on the curve.

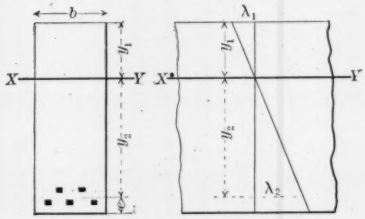


FIG. 8. FIG. 9. FIG. 10.

Fig. 8 is a cross-section of a steel-concrete beam of symmetrical strength.

Fig. 9 is the strain diagram showing the rectilinear variation of the strain across the section.

Fig. 10 is the stress diagram, showing the irregular variation in stress resulting from the uniform variation in strain across the section.

In addition to the symbols previously used,

Let f_t = tensile strength of the concrete, in pounds per square inch; P_t = total tensile strength of concrete for width b.

Then, as before,

$$P_c = \frac{5}{8} f_c \, b \, y_1 \qquad (13)$$

and for rock concrete,

$$f_c = \lambda_1 \frac{2 E_c}{3},$$

but

$$\lambda_1 = \lambda_2 \frac{y_1}{y_2}$$

* This statement would not be true in extreme cases. For example, we can conceive of a 30-in. concrete girder reinforced by a mere thread of fine wire. Here, the metal would have too little value to produce the condition, observed by Considère, wherein the metal reinforcement enables the concrete in tension to submit without loss of integrity to such great elongations. Probably the quantity of metal should never fall below an amount such that P_c would be at least equal to P_t. (See Fig. 10.)

Mr. Johnson.

and $$\lambda_2 = \frac{F}{E_s}.$$

Then $$\lambda_1 = \frac{F\,y_1}{E_s\,y_2}$$

and $$f_c = \frac{2\,F\,E_c\,y_1}{3\,E_s\,y_2}, \text{ or } y_2 = \frac{2\,F\,E_c}{3\,f_c\,E_s}\,y_1 \quad \ldots\ldots (14)$$

We also have for the steel:

$$P_s = \frac{F\,a^2\,b}{d} \quad \ldots\ldots (15)$$

For the concrete in tension:

$$P_t = \frac{8}{10}\,f_t\,b\,y_2 \quad \ldots\ldots (16)$$

The empirical constant, $\frac{8}{10}$, is obtained inferentially from the results of M. Considère. It cannot be much in error.

Then for the equation of the moments we have:

$$M_o = \frac{F\,a^2\,b}{d}\left(y_2 + \frac{2\,y_1}{3}\right) + \frac{8\,f_t\,b\,y_2}{10}\left(y_2 + \frac{2\,y_1}{3}\right) \quad \ldots\ldots (17)$$

The equality of forces on each side of the section gives:

$$P_c = P_t + P_s \quad \ldots\ldots (18)$$

or $$\frac{5\,f_c\,b\,y_1}{8} = \frac{8\,b\,f_t\,F\,E_c\,y_1}{15\,f_c\,E_s} + \frac{F\,a^2\,b}{d} \quad \ldots\ldots (19)$$

from which $$y_1 = \frac{120\,F\,a^2}{75\,f_c\,d - 64\,F\,d\left(\frac{f_t\,E_c}{f_c\,E_s}\right)} \quad \ldots\ldots (20)$$

or $$\frac{a^2}{d} = \frac{75\,f_c\,y_1 - 64\,F\,y_1\left(\frac{f_t\,E_c}{f_c\,E_s}\right)}{120\,F} \quad \ldots\ldots (21)$$

also $$y_1 + y_2 + e = t \quad \ldots\ldots (22)$$

The stress on the concrete under the bar is not considered in these equations.

For a given depth of beam, Equations (14) and (22) will give at once the values of y_1 and y_2, and then the value of $\frac{a^2}{d}$ can be obtained from (21), which, inserted in (17), will give the moment of resistance for a symmetrical beam.

For the case of a given external bending moment we obtain from (14) the relation y_1 and y_2, this relation being only dependent upon the character of the steel and concrete used. Inserting this value of y_2 in terms of y_1 in (17) we obtain an equation in which the known external bending moment is equal to a function of y_1 and $\frac{a^2}{d}$. The relation between these is obtained from (21), which will at once give the value of y_1 and y_2, and $\frac{a^2\,b}{d}$ for the given moment.

Mr. Johnson. To illustrate the operation of the formulas, and at the same time to show how greatly the design is dependent upon the modulus of elasticity as well as the crushing strength of the concrete, the writer has taken a problem with two cases involving different values for these functions.

Problem.—Required, a steel-concrete girder beam of equal strength above and below the neutral axis, 15 ft. between supports, capable of carrying, as an ultimate breaking load, 2 000 lbs. per linear foot.

Here, neglecting the weight of the beam,

$$M = \frac{12\ Wl}{8} = 675\ 000 \text{ inch-pounds.}$$

For Case I, let $E_s = 30\ 000\ 000$ lbs. per square inch.

$E_c =$ 4 800 000 " " " "
$F =$ 50 000 " " " "
$f_c =$ 1 500 " " " "
$f_t =$ 200 " " " "
$b =$ 6 ins.

Then from (14) we have,

$$y_2 = \frac{2 \times 50\ 000 \times 4.8}{3 \times 1\ 500 \times 30}\ y_1 = 3.55\ y_1;$$

also, (17) becomes,

$$M_o = M = \frac{50\ 000 \times 6\ a^2}{d}\left(3.55\ y_1 + \frac{2\ y_1}{3}\right) + \frac{1\ 600 \times 6 \times 3.55\ y_1}{10}$$

$$\left(\frac{3.55\ y_1}{2} + \frac{2\ y_1}{3}\right) = \frac{1\ 266\ 000\ a^2\ y_1}{d} + 8\ 300\ y_1^2.$$

$$\text{But } \frac{a^2}{d} = \frac{75 \times 1\ 500\ y_1 - 64 \times 50\ 000\ y_1\left(\frac{200 \times 4.8}{1\ 500 \times 30}\right)}{120 \times 50\ 000} = 0.00737\ y_1$$

Substituting this value of $\frac{a^2}{d}$ in the moment equation, we have,

$$M_o = 17\ 625\ y_1^2 = 675\ 000,$$

or $y_1 = 6.20$ ins.

Then $y_2 = 3.55\ y_1 = 22.0$ ins.,

and $t = 6.2 + 22.0 + 2 = 30.2$ ins.

$$\frac{a^2 b}{d} = 0.00737\ b\ y_1 = 0.274 \text{ sq. in., or slightly more than one,}$$

$\frac{1}{2}$-in. square bar.

We thus have the result that with the assumed values of the constants it will require a beam 6 ins. wide by 30 ins. deep, with one $\frac{1}{2}$-in. square bar imbedded, to carry 2 000 lbs. per running foot on a span of 15 ft.; which beam would be equally likely to fail by the crushing of the concrete above, or by the rupture of the concrete and metal below the neutral axis.*

* To this beam the exception made in the foot note on p. 122 would apply, and, in practice, more steel than this would have to be used. It serves the purpose of the present comparison, however.

Mr. Johnson.

For Case II, take $E_c = 2\,400\,000$ lbs. per square inch.
$f_c = \quad 2\,400$ " " " "
and all other values as before.

These values are by no means exceptional, it all depending upon the character of the materials used.

For this case,

$$y_2 = \frac{2 \times 50\,000 \times 2.4}{3 \times 2\,400 \times 30}\, y_1 = 1.11\, y_1,$$

instead of, as before, $3.55\, y_1$. This gives at once an idea of what may be expected.

$$M_o = M = \frac{50\,000 \times 6\, a^2}{d}\left(1.11\, y_1 + \frac{2\, y_1}{3}\right)$$
$$+ \frac{1\,600 \times 6 \times 1.11\, y_1}{10}\left(\frac{1.11\, y_1}{2} + \frac{2\, y_1}{3}\right)$$
$$= \frac{534\,000\, a^2 y_1}{d} + 1\,303\, y_1^2.$$

$$\frac{a^2}{d} = \frac{75 \times 2\,400\, y_1 - 64 \times 50\,000\, y_1 \left(\frac{200 \times 2.4}{2400 \times 30}\right)}{120 \times 50\,000} = 0.0264\, y_1$$

Substituting this value in the equation for the moment, we have:

$$M = 15\,403\, y_1^2 = 675\,000,$$

from which $y_1 = 6.62$ ins.
$y_2 = 7.35$ "
$e = 2.00$ "

and $t = 15.97$ " , or say 16 ins.

also $\frac{a^2 b}{d} = 6 \times 0.0264 \times 6.62 = 1.05$ sq. ins.

This, then, gives us a beam 6 ins. wide by 16 ins. deep, with one 1-in. square bar imbedded, as the equivalent of the beam in Case I, 6 ins. wide by 30 ins. deep, with one $\frac{1}{2}$-in. square bar.

Taking concrete at 25 cents per cubic foot in place, and metal at 3 cents per pound in place, and supposing the centering to be the same for each case, the cost of the first beam per linear foot would be 33.80 cents, and of the second, 26.86 cents. There are other conditions, also, which make the shallower beam preferable.

The foregoing will serve to show how important it is to know with what materials we are dealing, and it was for this reason that the writer in the first part of his discussion so strongly advocated determining the characteristics of the concrete for the particular material available on work of any consequence. To know the compressive strength, is not sufficient; it is just as important to know the modulus of elasticity in compression.

It is regretted that Mr. Wason did not give the center loads carried by the beams tested at the Massachusetts Institute of Technology, as

Mr. Johnson. the writer would have liked to apply his formulas to them; though, as the modulus of elasticity was not observed, it would not have been a very good test. From the fiber stresses which are given he has endeavored to argue back to this load, through Mr. Wason's premises, but it would seem that some error has been made in the calculation of these stresses, judging by the case of the ¼-in. rod, where both load and fiber stress were given.

Of course, the assumptions he has made in arriving at the fiber stresses are, as he himself says, erroneous, as are the formulas for the design of such beams. So, too, is the calculus discussion for the location of the neutral axis, it being based upon a uniform variation of stress across the section, which, as may be seen by reference to Fig. 3, is far from the truth.

Mr. Wason, in his work, in general, assumes the neutral axis to be half way between the center of the bars and the top of the beam. For the 12-in. beams tested at the Institute this gave the neutral axis for all beams, 7 ins. above the bottom. He says that by measurements taken on the beams, it was found that the position of the neutral axis varied from 4.74 to 10.2 ins. from the bottom, the mean being 7.06 ins., which he regards as a good check on the assumed location. One might as well say that the average number of inhabitants per square mile in the United States multiplied by the number of square miles in any given district, would give the population of that district. It might, but the chances would be very much against it.

The writer is disposed to regard the facts given by Mr. Wason as of much value, and hopes that he will supplement his discussion with further data with respect to the loads applied in the series mentioned, and other results of a similar nature.

The writer's specialty is not in the line of arch bridge construction; although he has worked up one or two designs for spans between 80 and 100 ft., which, however, were not built. These designs were for regular street service, the arch being from 10 to 12 ins. thick at the crown, with a rise of 1 in 8. Arches having a thickness of 3 ft. at the crown, with a span of 100 ft., such as recently witnessed by the writer, would be regarded by him as no application whatever of the steel-concrete principle of construction, the metal being practically thrown away.

Furthermore, the temperature stresses increase rapidly with the crown thickness, and are, naturally, severe where the latter is three times as much as it need be.

Mr. Parker. WILLIAM PARKER, M. Am. Soc. C. E.—The question of fire-proof construction has only been lightly touched upon in these discussions. It would seem that, as concrete is used so largely in fire-proof construction, more consideration should be given to the subject of the effect of fire on concrete, which, in the various fire-proofing systems,

is depended upon for strength, either when acting alone or in conjunction with metal. The metal which is depended upon for strength should be placed far enough from the surfaces exposed to heat, to insure the maintenance of a temperature which is not injurious to the strength of the metal and which will not cause undue expansion. Mr. Parker.

In the speaker's opinion, in some cases, this has not been done.

Just how far from the surface exposed to heat concrete will be seriously affected, and just what kind of concrete is best adapted for use on the surface, are the questions to be decided by the engineer or architect for the different conditions in his structure. So far as the speaker can learn, there have been but few carefully-made investigations to assist in deciding such questions, but it is his belief that concrete properly made, of either stone or cinders, and unprotected by plaster or otherwise, will not be destroyed by fire and water, such as it would be subjected to in a warehouse, more than 1½ ins. from the exposed surface, and this only when there is the concentration of a powerful fire-stream; and that after a depth of 2 ins. is reached there is no serious injury.

Cinder concrete is, no doubt, one of the best of non-conductors of heat, and, as it resists fire and water very successfully, it should be used whenever practicable.

These conclusions are from experiments on a small scale made by the speaker, and from observations of a few experiments on a large scale, as well as by reading what little he could find in print.

The thickness of concrete required for the protection of columns and long girders, particularly to prevent undue expansion, should be determined carefully by experiments, the temperature of both fire and beams or columns being recorded. The speaker can find scarcely a single record giving these data in a satisfactory manner.

The question of repairs should be borne in mind when designing. From experiments made by the speaker it seems practicable to replace a protecting layer of cinder concrete on a ceiling when it has become injured by fire and water. This question of repairs is one of special interest when considering the matter of insurance.

WILLIAM B. FULLER, M. Am. Soc. C. E. (by letter).—The use of combinations of steel and concrete in various forms of engineering construction is meeting with increased favor as engineers become better acquainted with its possibilities; but this form of construction, or the use of any formulas relating thereto, should not be attempted without adequate experience. Mr. Fuller.

It is, perhaps, very fortunate that the development of this industry has fallen into the hands of companies controlling so-called patent rights for the use of the various combinations, as these companies have learned by costly experience that there is concrete of all grades, and that it requires a high order of skill to produce good concrete,

Mr. Fuller. such as will enable them to guarantee safe work. The results attained by such skilled mechanics are far ahead of average work, and very large allowances should be made if such work is to be done by others than skilled workmen under skilled supervision.

The secret of success with steel-concrete construction is skill in the selection and putting together of the materials to form a monolithic mass, and, until the methods of manufacturing concrete become more systematic and the results more uniform, each worker in this field must depend in a large measure on his own experience as to the quality and strength of the work his method will produce.

It is particularly unfortunate that all engineers of the older school have taught that concrete must be placed dry, or, at least, so as to just quake under heavy ramming, and that the stone used should be of a uniform screened size, as success in steel-concrete work requires a radical departure from such teachings, and the use of light rammers, the placing of mushy concrete, and the use of a uniformly-graded aggregate.

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CIVIL ENGINEERING PRACTICE: SHALL IT BE REGULATED BY LAW?

An Informal Discussion at the Annual Convention, June 26th, 1901.

SUBJECT FOR DISCUSSION:

"Do the interests of the profession, and the duty of its members to the public, require that only those who are competent be allowed to practice as Civil Engineers?

"Under what authority, through what agency, and upon what evidence of competency, should applicants be admitted to the practice of Civil Engineering?"

By Messrs. SAMUEL WHINERY, FOSTER CROWELL, F. C. OSBORN, H. W. BRINCKERHOFF, FRANK A. HINDS, WILLIAM R. HUTTON, JOHN F. WALLACE, EDWARD A. BOND, J. P. A. MAIGNEN and SAMUEL WHINERY.

Mr. Whinery.

SAMUEL WHINERY, M. Am. Soc. C. E. (by letter).—Taking up these questions in the order stated, the first relates to the interests of the members of the profession to themselves and to each other.

The facts about the present situation are undisputed. Any man who chooses may assume to practice civil engineering. Without either special education or previous experience he may "hang out his shingle," advertise himself as a civil engineer, obtain clients, and collect fees for alleged professional services.

Mr. Whinery. That this condition of affairs is anomalous need not be argued. Neither in any other profession nor even in the well-established trades are similar conditions tolerated.

The preacher must be duly ordained by the proper authorities of the church to which he belongs before he is allowed to occupy a pulpit or pose before the world as a minister of the gospel.

The man who aspires to practice medicine must pass his examination and be regularly authorized before he can receive or treat patients.

No man is allowed to practice law before the courts until he has been admitted to the bar in the prescribed manner.

No one can teach in the public schools without a certificate of his or her fitness for such work, issued only after due examination. In many of the States no one can sell drugs without a license, based on his possession of the necessary qualifications for his duties. In the trades, while, as a rule, proper qualifications are not required by law, the members of each trade have attempted, and usually with success, to shut out the unqualified, through the agency of Trade Unions and other similar organizations. In this country, the great profession of engineering, in its several branches, is almost alone in being open to the pretender and the quack.

Do the interests of the profession of civil engineering require a change in this respect? The question may be viewed in two principal aspects. The first and least worthy may be called, to put it in its most sordid light, the selfish aspect. It is based on the right of self-preservation. In these days, to be a competent civil engineer implies a large amount of special education and training. Whether the education is acquired within or without the technical schools, does not matter. However acquired, it must be supplemented by training and skill, which can only be obtained through actual experience. Both the education and the experience cost money. They represent a large part of the capital invested in the business. It may fairly be claimed that this capital is equitably entitled to the same protection as is that invested in other lines of business. On mere commercial grounds alone, the man who invests money in any lawful business may reasonably expect and demand the protection of the law in all proper ways. The particular quality of the protection afforded will vary with the nature of the business. The merchant not only expects and receives protection from the thief and the burglar, but he is likewise entitled to protection from the nefarious competitor who sells goods under false names, or misrepresentations, or who appropriates his trade-marks.

May not the civil engineer who invests money or its equivalent in his professional business demand equal protection from the unfair competition of the quack who marks the goods he offers with counter-

feit labels, and thereby is enabled to steal a part of the business that rightfully belongs to the *bona fide* engineer? Even if it be admitted that the quack has the right to dispose of his goods, such as they are, he has no right to palm them off on the public under false trade-marks. The qualified members of the profession have the right to demand that his wares shall be sold for what they really are. Without any intention or desire to restrain fair and legitimate competition, may not engineers ask that competition shall be confined between lines which mark the limits of common honesty? Mr. Whinery.

The other aspect of the question is more attractive because it rests on higher grounds than mere self-protection or commercial necessity. Its basis is love for and loyalty to the profession. This is not the place or the time to enlarge on the usefulness and the dignity of that profession. Every true and worthy member of it holds it in the highest honor and is properly jealous of its good name. It is a name under which may be marshalled most of the world's greatest benefactors in the material progress of the human race. We are unwilling that its honor shall be tarnished or its usefulness limited by the actions of those who have no right to be numbered in its ranks, though they may assume its name. That the profession is being dishonored and its good name discredited by the actions of self-styled civil engineers, who have neither the necessary qualifications to practice it nor the common honesty to admit their incompetence, is a fact that every civil engineer knows so well that it is quite unnecessary to go into particulars. Doubtless every member of this Society could supply abundant facts and details from his own observation and experience to illustrate and enforce the truth of the statement. No one will assert that it is not the duty of every worthy and loyal member of the profession to protect its good name from this source of danger and dishonor. There should be, therefore, no dissent from the answer that the interests of the profession do demand that some effective method be adopted to prevent the incompetent from assuming to practice civil engineering.

To the second division of the question, relating to our duty to the public, the answer should be equally clear. The admitted facts are: That at present the public has no ready means of determining whether a man who poses as a civil engineer is competent or incompetent, and that, in the absence of such knowledge, it is often imposed upon by the charlatan.

Civil engineering, while it has been practiced throughout all ages, under other names, has been, until quite recently, without professional organization, and without well-defined limitations. The public, or a large part of it, has not been educated to appreciate either the natural or acquired qualifications necessary to constitute a competent civil engineer, or to discriminate between the real and the pretended.

Mr. Whinery. To it a civil engineer is simply a civil engineer very much as a plow is a plow; and, while it recognizes that some plows are better for special purposes than others, a field may be plowed well enough by almost any plow that can be picked up. The difference between good and bad engineering is not always obvious to those who themselves have not the qualifications to judge. The competent members of the profession are alone able, as a rule, to see and point out the blunders and the lack of professional good judgment of the pretender. That vast sums of money are squandered, promising enterprises wrecked, and reasonable expectations blasted every year through the operations of ignorant and incompetent civil engineers, are facts too well known to need proof or illustration. That we owe a duty to the public in this matter must be so obvious to every right-minded man in the profession as to require no argument. As well might it be contended that the physician owes no duty to the public when he sees a plague existing, or impending, which his professional knowledge and experience teaches him how to stop or prevent. From the standpoint of good citizenship alone, we have no right to stand silently by while our fellow-citizens are defrauded, or when their lives and property are threatened.

We now come to the second branch of the topic:

"Under what authority, through what agency, and upon what evidence of competency, should applicants be admitted to the practice of Civil Engineering?"

Here we arrive at the first ground, in the discussion of the topic, where there may properly be difference of opinion. Previous discussion has disclosed that the members of the profession are not at all agreed upon the best remedy for the admitted evils that exist.

It would be interesting to review what has been done or attempted to be done in this direction in other countries, though it is doubtful if the facts would be of very much value to us. The conditions abroad are, in many respects, so different from those existing in the United States, that it seems quite doubtful if the experience in other countries can be of much assistance to us. This is not said, however, to discourage the presentation, during the discussion, of interesting facts along this line.

The important requisites, in any scheme adopted for the regulation of the practice of civil engineering, are: That it should be simple, capable of efficient enforcement, and as free from the possibility of individual injustice as it can be made. In this last respect, it would be better to err in the direction of liberality than in that of unnecessary restriction. From the nature of our government, it is impossible to secure national action in the matter, and the several State legislatures alone must be looked to for the necessary legislation; and it must be recognized that nothing short of State legislative enactment will meet

the requirements of the case. It is very desirable that such enact- Mr. Whinery.
ments in the several States should be as nearly uniform as possible.

With these conditions in mind, the writer would suggest that the efforts of the profession be directed toward securing, in every State of the United States, the adoption of suitable acts regulating the practice of civil engineering. It is hardly necessary to say that the proper framing of such acts will be a matter of great importance, and the best talent of the profession, aided by good legal counsel, should have a controlling voice in their preparation.

The following outlines briefly the requirements which, in the writer's opinion, should be embraced in such a legislative act:

First.—It will be necessary to define accurately the meaning of the term "Civil Engineer" and to differentiate broadly and yet carefully the occupations which come under the head of practicing the profession. This will be by no means an easy task, as every one will recognize.

Second.—A State Board of Examiners should be created, whose duties will be to examine applicants for licenses to practice civil engineering within the limits of the State. This board may consist of five members, chosen from among men of recognized standing in the profession, three members constituting a quorum. They should be appointed by the Supreme Court of the State, in order to avoid, as far as possible, any political influence in the appointments. The members of the board should receive a reasonable annual salary, and their traveling and other necessary expenses should be paid out of the general funds of the State. Their term of office should be for a considerable number of years, say five, and the term of one member should expire each year. The board should meet at stated times and places, after due notice, to examine and license applicants.

Third.—The licenses issued by the board should be of three classes. Third-class licenses might embrace graduates from the civil engineering course of technical colleges of good standing, who have had little or no actual practice, and those applicants who have not graduated from such colleges, but who are found, upon examination, to be competent to survey lands, build common roads, inspect work in progress—in short, those who are just beginning the practice of the profession, and those whose ability and attainments justify practice in only the more simple work of the profession.

Licenses of the first class should be issued only to those members of the profession, who, by marked ability or long experience in any branch of civil engineering, have attained a good degree of eminence in the profession. Licenses of the second class should be issued to those intermediate between the third and first classes, as the law may designate, and as the examining board may determine. Licenses might be advanced from a lower to a higher class after due examination.

Mr. Whinery. This division of licenses into three classes, would be solely for the information of the public in assisting it to judge of the probable fitness of the licensees for contemplated services, a license of either class permitting the licensee to practice the profession without limitation.

The board should be authorized to accept the possession of a diploma from an engineering college of recognized good standing as satisfactory evidence of the technical qualification of applicants for third-class licenses, and to license at their discretion, without examination, applicants for first and second-class licenses, such engineers, of good character and standing in the profession, as may have been in the regular practice of the profession for not less than fifteen years before the passage of the act.

Licenses issued should be good until revoked by the board, but should expire on the permanent removal of the licensee from the State.

Fourth.—No person should be permitted to practice civil engineering in the State without a license duly issued by the board, and methods of procedure against violators of the law should be prescribed, and appropriate penalties provided; but persons duly licensed to practice in another State, under similar laws, should be exempted from securing an additional license under this act, upon filing a certified copy of such original license with the board. The law should not apply to members of the Engineer Corps of the U. S. Army, or to civil engineers of the U. S. Navy, when engaged in governmental work, but those engineers should not be allowed to engage in private practice without a license.

Fifth.—The board should be authorized to revoke any license for good cause, such as immoral or unprofessional conduct; or to substitute for a license of a higher class one of a lower class, for good cause shown.

Other requirements and details will be necessary and appropriate, but time does not allow the writer to mention them.

It has been suggested that membership in the American Society of Civil Engineers, or in other civil engineering societies of good standing, should qualify an engineer to practice; and it has also been suggested that the Society should have a voice in the examination and licensing of engineers. The writer would strongly oppose anything of this character, believing that such functions do not come within the province of the Society, and that it should scrupulously avoid any entanglement with matters of this character. The various societies, might, however, very properly participate in the adoption or inauguration of this or any other measure intended for the good of the profession, and for the benefit of the public.

In conclusion, the writer believes that the subject matter of this topic is of such great and far-reaching importance to the profession

that it justifies the appointment of a committee of the Society to consider the matter carefully in all its bearings, and report its conclusions to a future meeting. Mr. Whinery.

FOSTER CROWELL, M. Am. Soc. C. E.—The speaker believes that all will agree as to the desirability of regulating engineering practice; but, in regard to the method, he would not be inclined to favor the idea of licensing by different States. It should be recollected that the function of an engineer partakes more of a National than of a State character. In matters apart from engineering, the disadvantage of having different State requirements is apparent. As an illustration, it is known that this country is troubled by its diversity of laws in different States applying to the same conditions, exactly, of the citizen. Marriage and divorce laws and the medical laws are cases in point. In some States, a medical practitioner can obtain a full certificate on a very slender stock of preparation and ability, while in other States he cannot obtain it at all. If anything is to be done toward obtaining legislation, it should be of a National and not of a State character; and the requirements sketched by Mr. Whinery should be so broad and general that they could apply equally well to an engineer who went out to practice in the most important work in a new territory or a newly formed state as to one practicing at the center of civilization. The speaker is not inclined to believe, however, that much can be accomplished in that direction at the present period; and doubts very much whether the time is ripe to accomplish much through legislation. While this Society should not be the agent, yet the influence which the Society exerts, which is becoming greater from year to year, in stamping its members with its approval, will be the probable direction that the licensing of engineers will take. Not that a membership in this body is a license, by any means, because our Constitution requires that a man shall have practiced a number of years and become an engineer before he can become a member, nevertheless, the fact that the public appreciates that this Society has among its members only those whom it deems fit will perhaps be a solution of the question, and in the coming years render whatever legislation is necessary more easy. Mr. Crowell.

FRANK C. OSBORN, M. Am. Soc. C. E. (by letter).—To the first question the writer would answer, unqualifiedly, "yes." It would seem as though the engineering profession is as much entitled to protection as the professions of medicine and law. The responsibility of an engineer is certainly as great as in any of the other professions, and the standing of the profession would be improved by legislation similar to that at present in existence for physicians and lawyers. Mr. Osborn.

Not only the interests of the profession and the duty of its members to the public require this legislation, but the interests of the public itself demand the protection involved in such legislation.

Mr. Osborn. The second question is a much more difficult one to answer, and is one that should be thoroughly investigated before an attempt is made to answer it. The writer would suggest that a committee from the Society be appointed to investigate what has been done in other countries, and what is being done now. After this committee has thoroughly studied the question from every standpoint, they could then suggest to the Society what action should be taken. The matter is an extremely important one, and what is done should be done carefully and after mature consideration.

Mr. Brinckerhoff. H. W. BRINCKERHOFF, M. Am. Soc. C. E.—To the question "Do the interests of the profession, and the duty of its members to the public, require that only those who are competent be allowed to practice as Civil Engineers?" there can be but one answer. It is obviously to the interest of both the profession and the public that incompetent persons should not be allowed to practice as civil engineers. The real question is, "How are you going to exclude them, and them only?" and the latter part of the question is much the more difficult to answer.

There are two ways of excluding the incompetent: One is to forbid him to practice, and the other is to designate the competent men, or as many of them as wish to be so designated, and then let the public take its choice. This latter method may not, at first, appear to be so effective, but it affords the public a reasonable measure of protection while it largely avoids the danger of unjustly debarring anyone from securing work that he may be competent to perform. It must be remembered that it is much more difficult to determine in a general way the competence of a civil engineer than that of a member of any of the so-called learned professions. This is due not only to the vast field covered by civil engineering, which includes all branches except military engineering, but also to the many subdivisions of each branch, from designing to constructing, and even to operating; from the purely theoretical to the extremely practical; so that the engineer may be a scientist or a business man with large executive functions, and has sometimes to be a good deal of each. Then, too, as in most engineering work, a more or less extensive organization is required to carry it on; there must be some qualified to direct and plan and others who can only work under direction, and who, although in subordinate positions, may be very competent men.

It is obvious from this that it will be extremely difficult to define exactly the sphere within which any civil engineer may be competent, and outside of which he would be incompetent, and to do this in such manner as, on the one hand, to avoid injustice to the engineer by too narrow limitation, and, on the other hand, to avoid misleading the public by giving the engineer too great latitude.

Fortunately, as engineering is an exact science, it is comparatively

easy to judge of an engineer's competence by what he has already done. But, while we may know pretty well what he can do, we cannot pronounce with the same certainty as to what he cannot do, and are therefore more likely to err in attempting to define too closely the limits of his competence. This being the case, it would seem wiser, if any legislative control is to be attempted, for the State to content itself with certifying to the competence of the engineer within certain limits, without attempting to confine his practice within those limits. Then, if anyone wishes to employ a civil engineer for a particular work, he can easily find one who is competent for it, while if he wishes to take the risk of employing one who is not certified he is not debarred from doing so. To carry this plan out practically, probably the better way would be for the State merely to require that only those who hold its certificates of competency shall be employed on public works of any kind, or be permitted to prepare plans which require official approval. This would require only a slight extension of the present civil service machinery, and would tend to improve the engineering service of the State and municipalities, while it would afford an additional inducement for competent engineers to apply for certificates, which is important if the system is to command public confidence. Of course, certificates would be of various grades, and each would be limited to a special branch, though one engineer could hold certificates in more than one branch, and there should be provision for frequent change of grade, especially in the lower grades. Mr. Brinckerhoff.

As the legislation outlined herein is not prohibitory, any difference in the requirements of the different States would work no hardship or inconvenience, provided citizenship or actual residence was not made a condition of granting a certificate of competence. There would doubtless, at first, be much diversity in the requirements of the different States, but this, by trying so many different experiments simultaneously, would hasten the final solution, and the experiment could commence in one State without waiting for the consent of all. As no restrictions are imposed, the National government could also work along the same lines, while, if prohibitory legislation were required, it would seem to be beyond the power of the National government to enforce it in such a matter. Certainly, it has never attempted anything of the kind, and, fortunately, in the plan proposed it is not necessary.

It would seem to be entirely within the province of the American Society of Civil Engineers to recommend to the State and National governments such legislation as may be needed to carry these suggestions into effect, and they would certainly tend to "the advancement of engineering practice and the maintenance of a high professional standard," which are among the chief objects for which the Society exists.

FRANK A. HINDS, M. Am. Soc. C. E.—That there should be some sort of recognition, an examination and a certificate from a qualified Mr. Hinds.

Mr. Hinds. source, giving evidence, as far as possible, of a man's suitableness for the practice of civil engineering, seems to the speaker to be a desirable arrangement.

At present, we have only the degree granted by the engineering colleges, and usually at the end of the course of study.

A college education should have its due weight, but experience, ability and achievement should be the ruling motives. Many men have grown into the profession without college opportunities, and some of them are found among our best engineers.

If there could be an engineering rating for all men, based upon some similar or possibly better method than that now adopted in the admission of members to this Society, it would give an added dignity and attraction to the most noble profession of the age, and be an inspiration to young men to climb higher.

It would hardly do to have this controlled by each State government; nothing less sweeping than the general government of the United States could do it properly. If controlled by each of the States, there would be the same difficulty now encountered with regard to corporations, each State would have to recognize the diplomas of the other States, some one State would establish a low standard and pass candidates easily, and, as a consequence, would do a rushing business; but it would lower the standard, and, therefore, it should be national.

It would seem that the American Society of Civil Engineers has the ability, and is a very proper source from which such a plan might emanate.

Mr. Hutton. WILLIAM R. HUTTON, M. Am. Soc. C. E.—If the public is to be fully protected, as has been suggested, it will be necessary to have periodical examinations to define the grade of work for which each engineer may be competent.

The speaker sees nothing to be gained by the passage of this resolution. We must define what constitutes an engineer. If the Tredgold rule is applied, certain members of this Society, who are highly scientific men, will be excluded, as well as others who cannot be thus classed. Civil service examinations are a protection in the matter of public works. Promoters of private works rarely neglect to ascertain the qualifications of an engineer before engaging him. A certificate or license to practice would not be accepted without investigation as to competency.

Mr. Wallace. J. F. WALLACE, Past-President, Am. Soc. C. E.—The proposition seems impracticable. The inability to get uniform regulations in the different States, and, above all, the difference in the degree of our professional qualifications and in the requirements, make it so. The law and the principles of law are well understood. Lawyers can be licensed; ministers of the gospel can be licensed by their denomina-

tions, because all that is required is that they will subscribe to certain tenets and have a general education. Men in the trades can be licensed. But as to engineers, the qualities that underlie the successful engineer, and the character of his work are so varied and our profession demands such a variety of talent, that the speaker does not see that it is practicable to accomplish our object by such a license. If it is desired to engage a civil engineer for a particular piece of work, it would not be enough to employ one who simply had a license to follow the profession of engineering. It would be necessary to know the characteristics of the man, and whether he was adapted for the particular work in hand. It does not seem that the man, or the corporation, requiring particular qualifications for a particular place, would be assisted by a license system. Mr. Wallace.

EDWARD A. BOND, M. Am. Soc. C. E.—Our friend Mark Twain, in discussing the question of who should be allowed to practice medicine, said, relative to the proposed prohibition of the practice of certain kinds of treatment, certain Swedish movements, etc., that he never had had any inclination to be treated in that way until that discussion came up. When he found that he could not be treated that way he wanted to be treated that way. Now, in regard to the case of the civil engineer, it is questionable whether it is wise to try to secure legal formalities to keep certain men from attempting to practice that profession. In our country when a keen business man is in need of an engineer for any particular service, he sends for one, looks him in the eye, weighs him up, and in less than seven minutes knows whether he wants that man or not. He does not care whether that man has a license to practice the profession of civil engineering; but he judges of him by looking into his eye, by talking with him and finding out what his experience has been, and with that he is satisfied, and it is pretty sure that he is not going to make a mistake. What would a man do, for instance, in going to New York and being called, let us say, into the office of Mr. Harriman of the Union Pacific? Do you suppose he could pull the wool over Mr. Harriman's eyes, and be launched in the designing of some railroad or other large engineering work of which he would make an utter failure? It does not appear probable. That being the case, it is the speaker's opinion that we had better let the young men go on. If they can get a job of land surveying, let them have it. If they can step from that to the designing of a dam or a bridge, let them do it, and in that way they are inspired to study, not necessarily in a school, perhaps—all the better if they could study in school—but they are inspired to grasp such means of learning as they can, and then through that element of Americanism which all of us have, more or less, while they are at work they are studying that work at night, and in that way improving their condition, feeling that they are free to practice if they get the opportunity to practice. Mr. Bond.

Mr. Maignen. J. P. A. MAIGNEN, Assoc. Am. Soc. C. E.—The speaker does not know of the existence of any examining board in France to rate civil engineers. They are rated by the special school from which they come. If they come from l'École Polytechnique, they find their way into the army, navy, railway and other great administrations. If they come from l'École Centrale des Arts et Manufactures, they are known to be efficient in all branches of engineering, and are much sought after.

L'École des Ponts et Chaussées turns out engineers who have charge of roads, aqueducts, tunneling and other great enterprises. Darcy and Dupuit, who wrote on water supplies, came from this school. L'École des Mines fits men for mining operations. Lower down is l'École des Arts et Métiers, which turns out mechanical engineers.

It may be said, therefore, that in France it is the school that stamps the man.

Mr. Whinery. SAMUEL WHINERY, M. Am. Soc. C. E. (by letter).—The many members of this Society to whom this question seems to be one of great importance must regret that the subject was not more fully and generally discussed at the Annual Convention. The present is probably as favorable a time in the progress of the profession as will ever occur for the discussion of the subject and for arriving at a final decision in the matter. That the subject is one of importance to engineers seems to be generally conceded, and, if so, it merits the most careful consideration of every worthy and loyal member of the profession.

The appointment of a Committee to report on the subject at the next Annual Convention will keep the way open for further consideration and discussion, and it is hoped that the members of the Society will study for themselves the whole question in all its bearings, so that the Committee's report, when presented, may be intelligently discussed, and a wise course of action promptly decided upon.

In the discussion at the Convention the opinion was expressed that any legislation that might be thought advisable relating to the regulation of engineering practice should be enacted by the National Government rather than by the several State Governments. There can be no doubt that National legislation would be preferable if it were possible, but it must be remembered that legislation of the character contemplated is, by the Constitution of the United States, distinctly reserved to the several States, and that Congress has no authority to enact such laws. It follows, therefore, that if legislative control of professional practice is decided to be desirable, our efforts must be directed toward obtaining the necessary enactments from the several State legislatures, and we must do what we can to secure as great uniformity as may be possible in the provisions of the laws passed.

In view of the reference of the subject to a Committee, the writer does not deem it necessary to more fully review, at this time, the discussion at the Convention.

Vol. XLVI. DECEMBER, 1901.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 903.

THE DECOLORIZATION OF WATER.

An Informal Discussion at the Annual Convention, June 26th, 1901.

SUBJECT FOR DISCUSSION:

"The Decolorization of Water: When is it necessary? How may it be accomplished?"

By Messrs. GEORGE C. WHIPPLE, ALLEN HAZEN, GEORGE A. SOPER, GEORGE W. FULLER, J. P. A. MAIGNEN, J. N. CHESTER, F. P. STEARNS, DESMOND FITZGERALD and H. F. DUNHAM.

Mr. Whipple.

GEORGE C. WHIPPLE, Assoc. M. Am. Soc. C. E.—One of the essentials of an ideal water supply is that it shall be colorless. This condition is realized by most ground-waters and by some surface-waters, but, ordinarily, surface-waters possess an amount of coloring matter which varies according to the character of the water-shed and the length of time that the water remains in contact with organic matter. At the outset, it is important to distinguish between the terms "color" and "turbidity," for a water may be colorless and yet turbid, or it may be clear and yet colored. According to the strict definition now recognized by water analysts, the color of water is due to substances in solution, while the turbidity is due to substances in suspension. If the suspended particles are colored the water may, indeed, have an apparent color, but this is more properly described as

Mr. Whipple. "colored turbidity." Thus, the water of the Red River, in Louisiana, has a red turbidity; the water of the Schuylkill River has at times a black turbidity, due to coal dust; while water heavily laden with algæ may have a green turbidity. The true color, or vegetable stain, as it is sometimes called, is due, theoretically, then, to substances in solution; yet in practice this definition cannot be adhered to in all strictness, because many surface-waters contain suspended matter of organic origin in amounts too small to be measured as turbidity, but which are sufficient to affect the color. Practically, therefore, the color of a water may be considered as that part of the apparent color which remains after the visible turbidity has been removed.

Nature of the Coloring Matter.—The coloring matter in water is derived primarily from decayed vegetation—that is, from chlorophyll and its derivatives. It is usually brownish, but may vary in hue from greenish-yellow to reddish-brown, according to the nature of the substances dissolved. The exact chemical composition of chlorophyll is unknown, but it contains carbon, hydrogen, oxygen, nitrogen and iron, with traces of other elements. It is probable, therefore, that these constituents are all present in the coloring matter of water. It has been known for a long time that in an unpolluted water the amount of nitrogen, as albuminoid ammonia, and the amount of carbon, as represented by the oxygen consumed, were, within certain limits, proportional to the amount of coloring matter. These relations are shown by Figs. 1 and 2, which are based upon analyses of samples of surface-waters from various parts of the United States. In most cases the points represent yearly averages of monthly samples, and were selected with special reference to freedom from pollution, suspended matter and microscopic organisms. In a general way, the amount of iron increases with the color, but, inasmuch as the amount of iron is sometimes very small in high-colored waters, the relation is probably incidental rather than causal, so far as the depth of color is concerned. It seems likely, however, that iron exerts an influence on the hue of the water. Under certain conditions, moreover, iron plays an important part in the production of color. Ground-waters, for example, are often colored by ferric oxide, but, inasmuch as this is largely in suspension, the color produced is more properly classed as colored turbidity. The color which water acquires at the bottom of a deep pond is to a great extent due to the presence of compounds of iron.

Variations in Hue.—The colors imparted to water by extracts of different substances and by extracts of the same substance under different conditions vary slightly in hue. For example, the extract of birch leaves has a larger component of red than that of cedar, and the cedar extract is more red than that of maple. The color extracted

Mr Whipple.

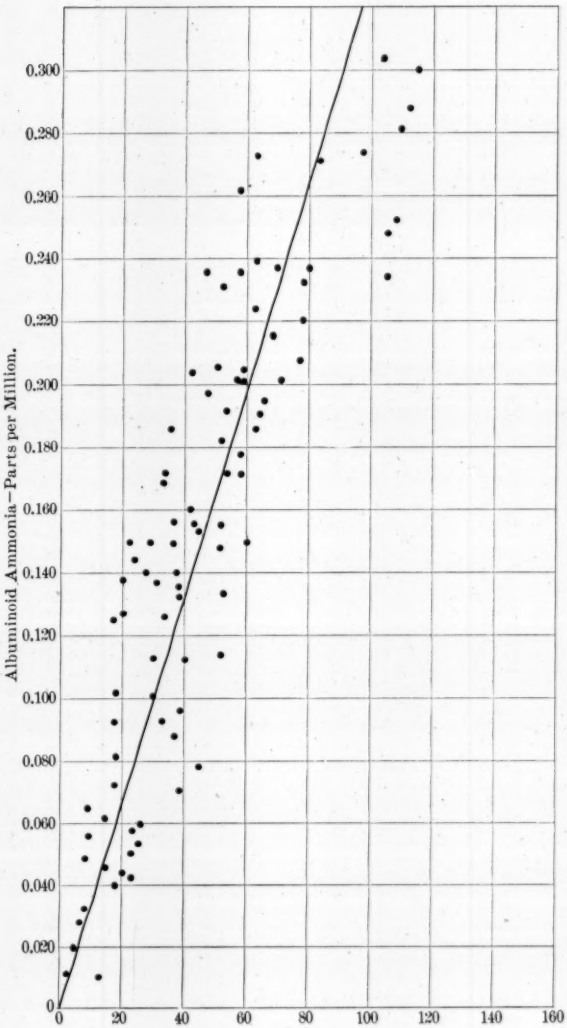


DIAGRAM SHOWING THE RELATION BETWEEN THE COLOR OF SURFACE-WATERS AND THE AMOUNT OF ALBUMINOID AMMONIA IN SOLUTION.

FIG. 1.

Mr. Whipple. from fresh peat is more green and less red than those of the leaf extracts mentioned. Old peat, however, has a larger component of red than fresh peat. There is some reason to think that these differences in hue are due, in part at least, to the presence of different amounts of iron. From analyses made by Dr. F. S. Hollis for the Boston Water-Works it appears that the amount of iron corresponding to a given amount of coloring matter decreases with subsequent extractions of the same substance, as shown by Table No. 1.

TABLE No. 1.

Extract.	Color.	Hue.	Iron. (Parts per million).	$\frac{\text{Iron.}}{\text{Color.}}$
Maple leaves, 8 days	37	Greenish-brown.	0.034	0.092
Maple leaves, subsequent extractions	20	Reddish-brown.	0.006	0.033
Elm leaves, 8 days	22	Greenish-brown.	0.016	0.073
Elm leaves, subsequent extractions	20	Reddish-brown.	0.011	0.055

The ferrous compounds of iron have generally a greenish color, and it seems likely that the greenish hue of fresh extracts may be due to such compounds. Experiment has shown that their addition to a colored water renders the hue more green. With subsequent extractions the iron becomes less in amount and perhaps changed to the ferric condition, causing an increase in the red component of the coloring matter. It is possible that manganese compounds may also exert an influence on the hue of water.

It is a matter of common observation that the hues of surface-waters change with the seasons. During the late autumn and winter the greenish-brown hues, due to the fresh extracts from newly fallen leaves, are most marked, while during the spring and summer the reddish-brown hues prevail.

The hue of a water is influenced by the amount of suspended matter present. The red rays are less readily absorbed than those of shorter wave-length; consequently, when suspended matter is present the proportion of red rays transmitted is relatively larger than would be the case if the water were clear.

It should be remembered that the hue of a water depends upon the depth of liquid through which the light passes, and that for the reason stated, namely, that the red rays are most easily transmitted, the hue becomes more red as the depth of the column of liquid increases.

Measurement of the Amount of Coloring Matter.—The Platinum-Cobalt Standard,* suggested by Allen Hazen, M. Am. Soc. C. E., may be fairly considered as the standard of color measurement. According to this standard as devised the color of a water was measured in

* *American Chemical Journal*, Vol. xiv, No. 4.

Mr Whipple

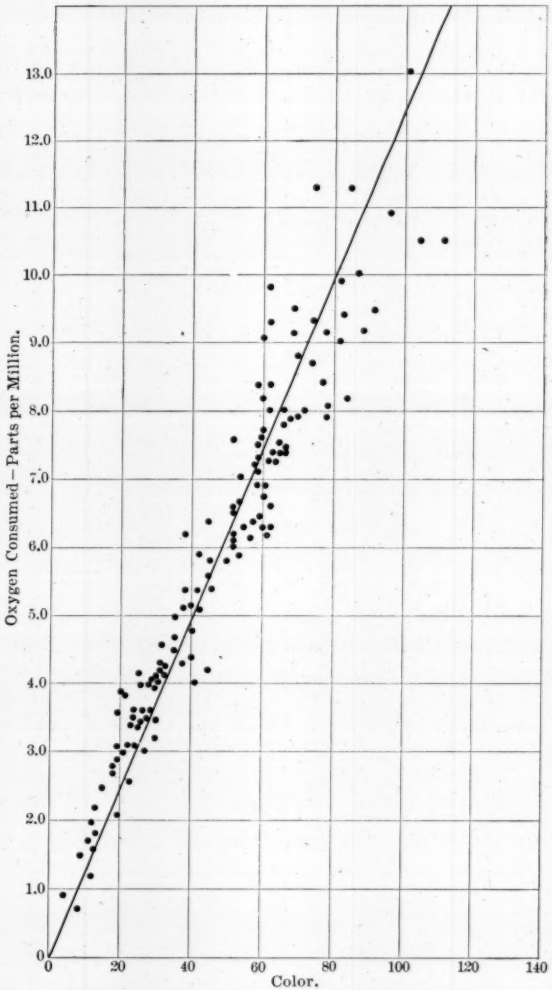


DIAGRAM SHOWING THE RELATION BETWEEN THE COLOR OF SURFACE-WATERS AND THE AMOUNT OF OXYGEN CONSUMED.

FIG. 2.

Mr. Whipple. terms of the amount of platinum in parts per ten thousand, which, in acid solution, with so much cobalt as matched the hue, produced an equal color in distilled water. The speaker believes that it will be a decided advantage, however, to make the unit of color equal to the amount of platinum in parts per million rather than in parts per ten thousand, in order that the figures which represent the color may be whole numbers rather than decimals, and in order that the determination may fall into line with the modern movement of expressing the results of analysis in parts per million. This method is used in this discussion. It will be observed that the figures correspond with those of the original method with the decimal point omitted. A stock solution, which has a color of 500, is made by dissolving 1.246 grams of potassium-platinic chloride ($PtCl_4.2KCl$), containing 0.5 gram platinum, and 1 gram of crystallized cobalt chloride ($CoCl_2.6H_2O$), containing 0.25 gram of cobalt, in water with 100 cu. cm. of concentrated hydrochloric acid, and making up to one liter with distilled water. By diluting this stock solution, standards are prepared containing 0, 5, 10, 15, 20, 25, 30, 35, 40, 50, 60 and 70 parts of platinum per million, which are kept in 100-cu. cm. Nessler tubes 29 cm. high, 2.2 cm. in diameter, with the 100-cu. cm. mark 23.5 cm. above the bottom. The color of a sample is observed by filling a similar tube with the water, comparing it with the standards, and noting the point where the colors match. The observation is made by looking vertically downward through the tubes upon a white surface placed at such an angle that light is reflected upward through the columns of liquid. A still more accurate method of making the comparison is by the use of a colorimeter like that devised by Fitz-Gerald and Foss.*

Some objection has been raised to the Platinum-Cobalt Standard, because with high-colored waters it is difficult to match the hues. If, however, waters which have a color above 60 are diluted with colorless water before the comparison is made there is no trouble from this cause.

The Nessler Standard and the Natural-Water Standard, which was made to correspond with it, have been largely used in the past, and are still adhered to upon some important investigations where it is desired to keep the records continuous. In order to facilitate comparison between these methods and the Platinum-Cobalt Standard, Table No. 2 may be found convenient. It was prepared at the Chestnut Hill Laboratory, of the Boston Water-Works, under the direction of Desmond FitzGerald, Past-President, Am. Soc. C. E.

Amount of Coloring Matter in Surface-Waters.—The amount of coloring matter in surface waters varies within wide limits. Some waters are almost colorless, while others have a color as high as 500

Journal of the Franklin Institute, Vol. cxxxviii, December, 1894.

Mr. Whipple.

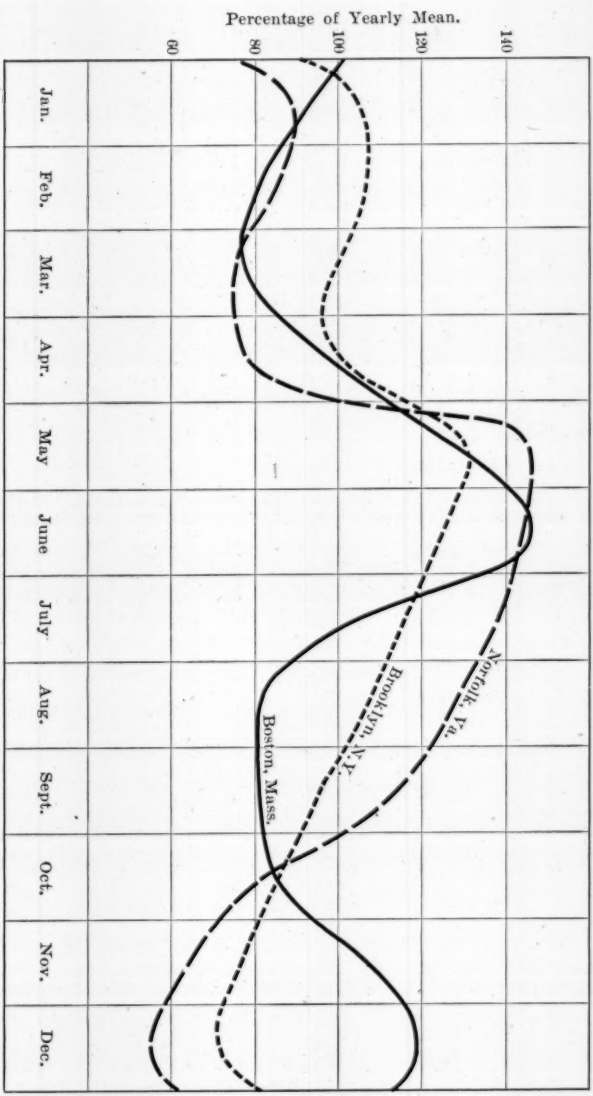


DIAGRAM SHOWING THE SEASONAL CHANGES IN THE COLOR OF WATER AT BOSTON, MASS., BROOKLYN, N.Y., AND NORFOLK, VA. EXPRESSED IN PERCENTAGES OF THE YEARLY MEAN.

FIG. 3.

Mr. Whipple.

TABLE No. 2.—For Transforming Color Readings from the Nessler or Natural-Water Standard to the Platinum-Cobalt Standard.

	Color of the Water in Parts per Million of Platinum.									
Nessler or Natural-Water Standard.	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.00	0	2	4	6	8	9	11	13	15	17
0.10	18	19	20	20	21	22	23	24	24	26
0.20	26	27	27	28	29	29	30	31	32	32
0.30	33	34	34	35	35	36	37	37	38	38
0.40	39	40	40	41	42	42	43	44	45	45
0.50	46	47	47	48	48	49	50	50	51	51
0.60	52	53	53	54	54	55	56	56	57	57
0.70	58	58	59	59	60	60	61	61	62	62
0.80	63	64	64	65	66	66	67	68	69	69
0.90	70	71	72	73	74	75	77	78	79	80
1.00	81	82	82	83	84	84	85	86	87	87
1.10	88	89	89	90	91	91	92	93	94	94
1.20	95	96	96	97	98	98	99	100	101	101
1.30	102	103	103	104	105	105	106	107	108	108
1.40	109	110	110	111	112	112	113	114	115	115
1.50	116	117	117	118	118	119	120	120	121	121
1.60	122	123	123	124	125	125	126	127	128	128
1.70	129	130	130	131	132	132	133	134	135	136
1.80	136	137	137	138	139	139	140	141	142	142
1.90	143	144	144	145	146	146	147	148	149	149
2.00	150	....	....	....	....	....	....	....	....	....

or even 1 000. These extremely high-colored waters, however, are found only in swamps, and are usually confined to small pockets where the water lies stagnant. The amount of coloring matter in a surface-water depends chiefly upon the character of the vegetation on the water-shed, and upon the length of time that the water remains in contact with the organic matter, but other factors enter into the problem. If water is collected from a water-shed which is rocky, with few trees and with steep slopes, the color will be ordinarily below 20; if collected from a wooded area or from cultivated land with steep slopes it may be from 20 to 50; if from a wooded or cultivated area with gentle slopes it may be from 50 to 100; and if from an area upon which there are numerous swamps it may range from 100 to 500 or more. These figures are only approximate, but they give a general idea of what may be found under different conditions. In most cases the problem is complex, and the color of the water collected from the water-shed is the component of numerous small feeders which drain areas which differ in character, and which, consequently, contribute different amounts of coloring matter.

In New England, where the color of the surface-waters has been studied more minutely than elsewhere, many of the public water supplies have an average color of more than 50. In general, it may be said that high-colored waters are most common in the Northern States. They are, of course, found elsewhere, and wherever swamps

exist, but in regions south of the glacial drift many of the streams are clay-bearing, and the presence of clay tends to reduce the amount of coloring matter in solution, as will be mentioned later. In the Northwestern States the colors are often very high. As a rule, high-colored waters are found in comparatively small streams which drain limited areas, but there are exceptions to this. Some of the large rivers in Maine carry very dark water; the Mississippi River at Minneapolis often has a color above 100. In the Northern States, where logs are floated down the streams and remain for months in the water, and where saw-dust is washed into the streams in great quantities, some of the coloring matter may be traced to this source. Mr. Whipple.

The amount of color in the water of any stream is subject to seasonal variations dependent upon the changes that take place in the foliage and upon the distribution of rainfall. In the Northern States the color of the water in the streams during the winter and early spring is usually at its minimum, because the ground is frozen and the water and melted snow pass into the stream without coming in contact with the organic matter on the ground, and because the dark-colored water lying in shallow swamps is frozen and does not contribute to the flow. The spring rains, however, cause the swamps to overflow, and the dark water from these sources causes an increase in the color of the streams. During the summer, when the swamps dry up and the streams become low, there is a decrease in color in many streams, but where the drainage from the swamps is continuous there may be a constant increase. In the autumn, after the leaves have fallen, there is often a marked increase in color. In the small streams of the Boston Water-Works, where continuous observations cover a long series of years, it was found that the seasonal changes in color were substantially as shown in Fig. 3. In Brooklyn, N. Y., where the cold weather is less severe, the winter minimum is not as low as at Boston, the spring maximum occurs earlier in the season, and the second maximum occurs in January instead of in December. At Norfolk, Va., the curve is similar to that at Brooklyn, but the fluctuations in color are somewhat greater. In large streams, and in lakes and storage reservoirs, the seasonal variations are not as clearly marked as those indicated, but they agree in their general character.

Rainfall has a marked influence on the color of small streams. Weekly observations on a small stream on Long Island, covering a period of three years, showed that the color was almost directly proportional to the amount of rain. This stream was fed directly from a swamp. Observations on the streams of the Metropolitan Water-Works gave similar results.

Color Due to Stagnation in Deep Ponds.—No better illustration of the effect of stagnation on the color of the water in a deep lake can be given than that of Lake Cochituate, in Natick, Mass., where weekly

Mr. Whipple. observations of color have been carried on since 1891. Some of these results are given in Fig. 4. During the summer the stagnant water at the bottom becomes highly colored, as shown by the full line, and when, at the time of the autumnal overturning, this dark water is carried upward, it tends to increase the color of the upper strata, as may be seen from the diagram. The same phenomenon takes place during the period of winter stagnation, but on a smaller scale. The explanation for the increase of color at the bottom during stagnation is as follows: At the bottom of the lake there exists a deposit of organic matter, and in the oxidation of this the oxygen dissolved in the lower layers becomes exhausted. As none can be secured from above, decomposition progresses under anærobic conditions, and cannot be carried to its final state of complete oxidation until the overturning. Under these conditions iron is taken into solution in the ferrous condition, and, together with dissolved organic matter, causes the increased color. On exposure to the air the iron becomes oxidized to the ferric condition causing a further increase in color, and a change in the hue from greenish-brown to reddish-brown. The dotted line in the diagram shows the average color of the four principal inflowing streams, obtained by combining the results of each according to the size of their drainage areas. It represents, however, only a part of the water which enters the lake.

Amount of Color in Ground-Water.—Ground-water is usually colorless, or practically so. Wells sunk in swamp areas, however, may, under certain conditions, yield a water which is high-colored. In passing downward through the surface layer of peaty matter, or through deeper deposits of organic matter, the oxygen may become exhausted, and organic matter and iron be taken into solution, as in the case of a stagnant pond. On exposure to the air the iron in such a water may become oxidized and the color of the water increased. To a considerable extent the color is due to iron oxide in suspension, but, even after filtering through a Pasteur filter, some coloring matter may remain.

Even where no organic matter is present, a ground-water may contain a large amount of iron in the form of ferrous carbonate, which on exposure to the air becomes oxidized to the ferric condition, with the formation of a yellowish-brown precipitate.

Decolorization of Water by Natural Processes.—The color of the water in large lakes is seldom high. There are three factors which tend to produce this result: Dilution, precipitation and the bleaching action of sunlight.

Most large lakes receive water from a water-shed which varies in character. Certain swamps and wooded areas may contribute dark-colored water, but usually, there are other areas which furnish water of low color; at times when the surface of the lake is drawn down to a

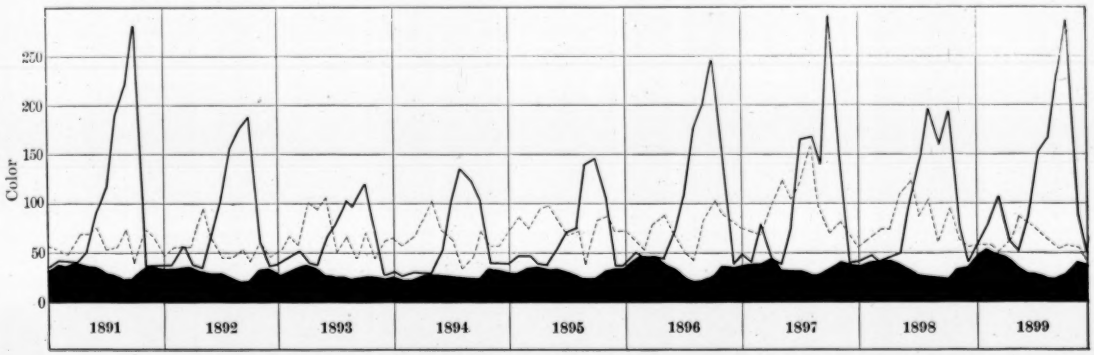


DIAGRAM SHOWING THE COLOR OF THE WATER IN LAKE COCHITUATE.

THE SHADED AREA SHOWS THE COLOR AT THE SURFACE;
THE FULL LINE SHOWS THE COLOR AT THE BOTTOM;
THE DOTTED LINE SHOWS THE AVERAGE COLOR OF FOUR INFLOWING STREAMS.

FIG. 4.

Mr. Whipple. level below that of the water table the color may be materially reduced by dilution with colorless ground-water. In order to estimate what the color in a lake or reservoir will be it is necessary to take account of all the water that enters and to give each source weight in proportion to its color and volume of flow; it is also necessary to allow for the amount of coloring matter that leaves the lake. As the flow of streams varies greatly at different seasons of the year, any calculation, to be accurate, must be based upon observations extending over at least one year, with observations made as often as once each week. If there were no other factors to be taken into account the calculated color obtained by this method would agree exactly with the observed color of the water in the lake. The observed color, however, is usually lower than the calculated color because of decolorization due to storage.

Unless waters have a noticeable turbidity, color readings are generally made without first filtering the samples, and experiments have shown that even where there is no evident turbidity surface-waters taken from swamps often contain enough finely-divided suspended organic matter to materially influence the color. Upon standing, some of this suspended matter will settle to the bottom, as may be observed in sample bottles, with consequent diminution in the apparent color of the supernatant water. In lakes the loss of color from this cause is small, but yet appreciable.

A more important factor in the decolorization of water in storage reservoirs is the bleaching action of sunlight. F. P. Stearns,* M. Am. Soc. C. E., has stated that in an unused reservoir 20 ft. deep the color of the water decreased from 40 to 10 (Nessler scale) in six months. A study of the nature and amount of the bleaching of colored waters was made by FitzGerald and Foss† by exposing bottles of high-colored water to sunlight for known periods under different conditions. It was found that the reduction of color was about 20% for each 100 hours of bright sunlight, and that all the color could be removed by sufficient exposure. It was also found that the actinic rays were most efficient in removing the color. Samples exposed in blue and white bottles showed much greater reductions than samples of the same water exposed in yellow and red bottles. The bleaching action was influenced but slightly, if at all, by temperature, and the part played by sedimentation was comparatively small. For equal periods of exposure to sunlight, the amount of color reduction was the same at all seasons of the year.

In order to determine the amount of bleaching action that takes place in actual practice the speaker once made a series of experiments

* "Special Report on Examination of Water Supplies," Massachusetts State Board of Health, 1890.

† Annual Report, Boston Water-Works, 1894.

in which bottles of colored water were exposed at different depths in reservoirs under different conditions. The results of three of these experiments are shown in Table No. 3. Mr. Whipple.

TABLE No. 3.

	Experiment No. 1.	Experiment No. 2.	Experiment No. 3.
Color of water in reservoir	20	37	44
Time of exposure, number of days	29	30	32
Color of water exposed	175	270	170
Percentage of reduction of color—			
In the air	.....	52%	41%
At depth of 0.5 ft.	65%	29%	20%
" " 1.25 "	32%	18%	12%
" " 2.5 "	21%	4%	4%
" " 5.0 "	14%	4%	3%
" " 7.5 "	3%	0%	0%
" " 10.0 "	1%	0%	0%
" " 15.0 "	0%	0%	0%

It was found that at the surface the bleaching action was considerable, but that it decreased more or less rapidly with increasing depth, according to the color of the water in the reservoir. In very dark waters the bleaching action is probably confined to a layer near the surface not over 1 ft. thick, but as the water becomes lighter, the action takes place at greater depths, consequently the color removal proceeds at an accelerated rate.

The fact that ground-water is colorless is due to the action of the soil, and especially to the aluminum compounds there present. These act chemically upon the coloring matter, as will be pointed out later. The decolorizing effect of aluminum compounds is also observed in turbid waters which carry clay in suspension. In such waters as those of the Ohio, the Allegheny, the Potomac, etc., there is very little dissolved coloring matter, and when the turbidity has been removed the waters are almost colorless as well as clear. The effect of clay upon the dissolved coloring matter may be illustrated by the following experiment. Kaolin in various quantities was added to bottles of colored water; the bottles were shaken and the water passed through a Berkefeld filter. The colors before and after treatment were as follows:

Original color.....68
0.5 gram kaolin added to 250 cu. cm. water.....52
1.0 " " " " " 47
1.5 grams " " " " 37

Amount of Coloring Matter Permissible in a Water Supply.—As far as can be learned, the organic matter which gives color to water is in no way injurious to health. In the light of modern bacteriology, it is almost

Mr. Whipple. certain that the intestinal disorders formerly attributed to swamp-water are due to other agents than the coloring matter, though these agents may accompany the coloring matter. The subject of color is to be considered, therefore, not from a sanitary, but from an æsthetic point of view. No one will deny that a brown-colored water is less attractive than a colorless water, and in these days, when the public is demanding improvements in the quality of water, and when standards of quality are becoming crystallized, this characteristic must be taken into account. Moreover, certain industries, such as the manufacture of paper and certain fabrics, bleacheries, etc., are said to demand a water that is both colorless and clear.

At present there is no recognized standard of permissible color. People from different localities are influenced by the character of the water to which they are accustomed. To a person who has been used to well-water, the water from a stream which had a color of 20 would seem dark, while another person used to swamp-water would consider it light. Below 20, however, it is seldom that the color of a water would be complained of. Such a water has a noticeable color when viewed at some depth in a porcelain-lined bath tub, or even in a carafe upon a white table-cloth, but in a glass tumbler it would scarcely be noticed. Between 20 and 40 the color might or might not be regarded with disfavor, according to the tastes of the individual, but above 40 it would be quite generally regarded as "high-colored," or "swampy." Perhaps 30, as an average of the foregoing figures, may be regarded as a fair limit for the permissible color at the present time.

To what extent a city or town is warranted in expending money for the mere removal of coloring matter depends upon the tastes of the citizens and the financial standing of the community. It is not to be looked upon as a necessity, but as a luxury. It should be recognized, however, that a good water supply is not only a matter of civic pride; it is of financial value, and especially so to a residential community. Now the general public judges a public water supply largely by its appearance, hence any reduction in the amount of coloring matter is really adding to the attractiveness of a city or town as a place of residence.

Methods of Decolorization.—The methods used for reducing the color of a water supply may be classified as follows:

1.—Prevention of color, by drainage of swamps and removal of organic matter from reservoir sites.

2.—Reduction of color by filtration without the use of chemicals.

3.—Decolorization by the use of chemicals followed by filtration.

4.—Decolorization by the use of chemicals without filtration.

Prevention of Color by the Drainage of Swamps and by the Removal of Organic Matter from Reservoir Sites.—It will be sometimes more economical to prevent surface-waters from acquiring color than to remove

the color by any of the processes mentioned. The color of the water in any stream or lake is the component of the colors of its various feeders. It is seldom that these feeders are equally colored, and it is often the case that some one (or some few) of them furnishes most of the coloring matter. By dividing a gathering-ground into parts, and by estimating the color and yield from each, the effect of any particular feeder upon the entire yield may be calculated. Steps may be then taken to drain the swamps and to intercept the water of low color from upland districts and lead it into the streams without allowing it to enter the swamps. The location of ditches and canals for these purposes must depend upon local conditions. Sometimes a comparatively small expenditure of money will yield most excellent results. Desmond FitzGerald, Past-President, Am. Soc. C. E., who was the first to attempt work of this kind upon a large scale, estimated that by an expenditure of $12 000 the color of the water furnished by the Ashland Reservoir of the Metropolitan Water-Works, Boston, would be reduced from 56 to 45. This was equivalent to $44 per acre of swamp, or $8.57 per million gallons of storage capacity. He also estimated that, by digging a system of canals, the water from the Cedar Swamp district, embracing an area of 20 sq. miles, could be reduced from 106 to 60 at an expenditure of $250 000. Mr. Whipple.

By removing the organic matter from the bottoms of reservoirs, this source of color is eliminated. When the amount of peat is large and the reservoir is comparatively shallow, the color acquired may be considerable and the action may be constant. With deep reservoirs, where stagnation occurs, the color will be yielded intermittently, as shown by Fig. 4. With clean reservoirs, storage of water will reduce its color instead of increasing it.

Since the color of a water may be permanently lowered by the proper treatment of the water-shed, and as the cost of decolorization by the use of chemicals increases with the amount of coloring matter present, it is evident that instances may be found where the former method will be the cheaper, and that, even where some form of decolorization is resorted to, the annual charges may be reduced by decreasing the color of the raw water by drainage of the swamps.

Reduction of Color by Filtration Without the Use of Chemicals.—Quartz sand has absolutely no effect upon the coloring matter in water. Simple sand filtration, therefore, cannot be depended upon to render a water colorless. Yet sand filters do remove a portion of the color, and for the following reasons:

It was stated that natural waters owe a portion of their color to finely-divided organic matter held in suspension. This suspended matter is removed by filtering through a Berkefeld filter, which is composed of silica, or through a bed of sand. Consequently, the color is somewhat reduced. A second filtration through these media will not

Mr. Whipple. further reduce the color, showing that the action is merely a mechanical one.

The sand used for filter beds usually contains a certain small amount of clay, and as the aluminum compounds have the power of uniting with the coloring matter, there is a slight decolorization from this cause. This action is greatest in a new filter, because, ultimately, the clay becomes largely removed. Even when no clay is present in the sand it is sometimes carried in suspension by the water and deposited on the top of the filter, where, to a slight extent, the same action may take place.

The oxidation of the coloring matter in the filter is probably insignificant in amount.

The Pasteur filter is sometimes used to filter samples of water before the color readings are made, but the aluminum compounds used in the construction of this filter exert a decolorizing action upon the water, just as does the clay in a sand filter. For this reason a Berkefeld filter, which is composed almost wholly of silica, is to be preferred.

If a water is filtered through animal charcoal, or bone char, it may be completely decolorized. This has been explained by saying that the coloring matter was oxidized in the pores of the char, where it came into intimate contact with the oxygen. Experiments, however, have apparently indicated that it is the calcium phosphate present in the bone char which is the real decolorizing agent. Animal charcoal may serve to decolorize water when it is to be used for industrial purposes, but there are objections to its use for filtering a domestic supply. It soon becomes foul and acts as a breeding place for bacteria to such an extent that the bacterial contents of the water filtered may be increased by several hundred per cent.

Decolorization by the Use of Chemicals Followed by Filtration.— Before the coloring matter can be removed from water it must be changed chemically. To bring about this change sulphate of alumina is commonly used in connection with filtration, generally by the so-called Mechanical, or American system. According to the accepted theory of the action of aluminum sulphate as a coagulant, the salt is decomposed by reaction with the carbonates* present in the water, with the formation of sulphates of the alkaline earths in place of their carbonates and of aluminum hydrate, which is the effective agent of coagulation. The aluminum hydrate then "drags down" the coloring matter, forming a flocculent precipitate, or insoluble lake, which is removed by filtration. If the carbonates present are not sufficient in amount to decompose all the aluminum sulphate added, the hardness of the water must be artificially increased in order that no undecomposed coagulant may remain in the water. As a general proposition, this is

* The determination of "alkalinity" gives the amount of carbonates and bicarbonates present.

Mr. Whipple.

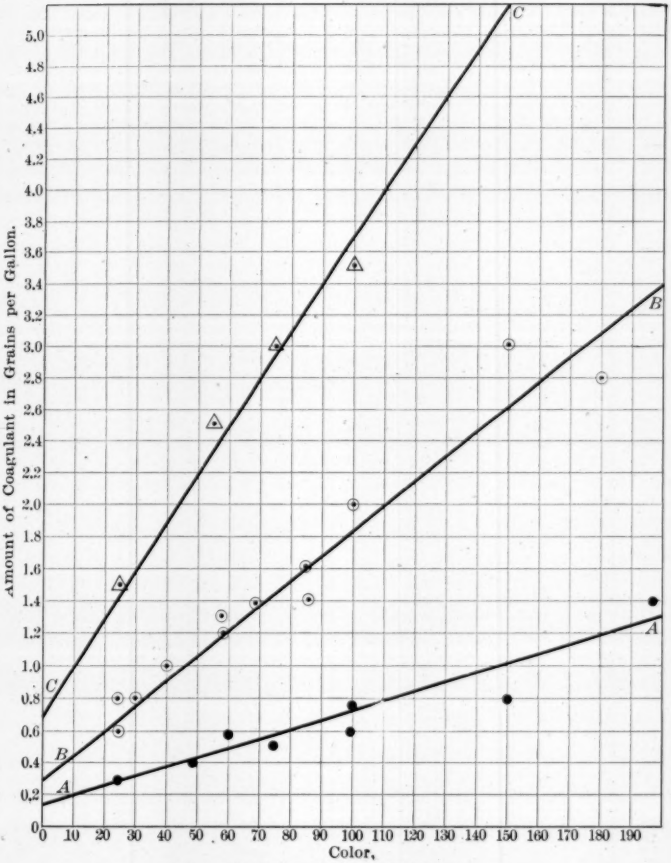


DIAGRAM SHOWING THE AMOUNT OF SULPHATE OF ALUMINA REQUIRED TO DECOLORIZE WATERS OF DIFFERENT DEPTHS OF COLOR, WHEN THE COAGULANT IS ADDED IN DIFFERENT WAYS

LINE A A = ALUMINUM SULPHATE ADDED TO COLORED WATERS WHICH HAD NO ALKALINITY.

LINE B B = ALUMINUM SULPHATE ADDED TO COLORED WATERS OF MODERATE ALKALINITY.

LINE C C = ALUMINUM SULPHATE CHANGED TO HYDRATE BEFORE ADDING TO COLORED WATERS.

FIG. 5.

Mr. Whipple. perfectly true, but experiments made by the speaker have shown that so far as decolorization alone is concerned an increase in hardness is not desirable, and, further, that the complete absence of carbonates is favorable to economical decolorization. To illustrate:

A water which had a color of 100 on the platinum scale, and an alkalinity of 20 parts per million, required 2 grains of aluminum sulphate* per gallon to decolorize it, while a water of equal color but with no alkalinity required only 0.7 grain. This indicated that the formation of aluminum hydrate was not necessary for the removal of color, and that there was a direct reaction between the aluminum sulphate and the coloring matter. A series of dark-colored waters free from carbonates was then prepared by diluting an extract of maple leaves in distilled water. To each of these differently colored waters in 1-liter bottles aluminum sulphate was added in various amounts, and the exact amount necessary to decolorize was determined by allowing the water to stand 24 hours and then filtering through paper. The results of these determinations are shown in Fig. 5 by the line *A A*. It will be noticed that the amount required was proportional to the color of the water. The experiments were then repeated, using an extract of birch instead of maple, with practically the same result. A natural water was then rendered neutral to methyl orange by adding enough hydrochloric acid to neutralize the carbonates, and here, again, the same results were obtained.

To find the amount of aluminum hydrate required to produce decolorization, the aluminum sulphate was changed to hydrate before adding it to the water. This was done by putting known amounts into colorless, hard waters, and, after allowing them to stand, mixing with the colored waters. It was found that when so used a very much larger amount of sulphate was required than when the direct reaction was obtained. The amount of aluminum sulphate required to decolorize waters of different colors when first changed to hydrate is shown in Fig. 5 by the line *C C*. In general, it is about five times as much as that shown by the line *A A*.

In actual practice, when aluminum sulphate is added to a colored water which contains carbonates, it seems probable that both reactions take place simultaneously, and that a part of the aluminum sulphate reacts directly with the coloring matter, while the remainder is decomposed by the carbonates, the aluminum hydrate thus formed also reacting with the coloring matter. To show this, a series of experiments was made upon colored waters which had an alkalinity of 40 parts per million. The aluminum sulphate was added to the water in 1-liter bottles and shaken thoroughly and immediately. The results are shown by the line *B B*. It will be seen that this line occupies a mean position

*The aluminum sulphate used in all the experiments contained 17% of available alumina.

between the lines *A A* and *C C*. This is the condition met with in practice, and the results agree fairly well with those which have been obtained with mechanical filter plants. Hazen* has stated that : Mr. Whipple.

"A removal of 80 to 90% of the color can be effected by the use of a quantity of sulphate of alumina equal to rather more than 2 grains per gallon for waters having colors of 1.00, * * * and proportionate quantities for more and less deeply colored waters."

According to the line *B B*, 1.8 grains per gallon were required to decolorize a perfectly clear water which had a color of 100, which is but slightly below Hazen's estimate.

While the two reactions take place simultaneously, their relative importance in decolorization varies according to the alkalinity of the water, and according to the rapidity with which the coagulant is mixed with the water. To illustrate:

Two bottles of clear water which had a color of 100 but no alkalinity were treated with 0.8 grain of aluminum sulphate per gallon. The first was shaken immediately, while the second was allowed to stand one hour before shaking. At the end of 24 hours both samples were decolorized. Two bottles of clear water which had a color of 100, but an alkalinity of 50 parts per million, were similarly treated with 2 grains of aluminum sulphate per gallon. After 24 hours the sample which was shaken immediately was decolorized, while the sample that stood one hour before shaking was not. To the latter it was necessary to add 3 grains per gallon before the color was removed. In this case it would appear that the aluminum sulphate was not at first disseminated through the entire body of water, but became in great measure changed to hydrate before the mixing took place; the hydrate formed was not as effective in removing the color as the sulphate acting directly, consequently a larger amount was required. It is evident, then, that for economical decolorization provision should be made for rapidly mixing the applied chemical with the water, so that it may come in contact with the organic matter before it has all changed to hydrate. It is further evident that the harder a water is the more difficult it is to treat, and the steeper will be the slope of the line *B B*. By neutralizing a water, or by reducing its alkalinity before the aluminum sulphate is added, the amount of chemical necessary to produce decolorization may be very materially reduced.

The direct reaction between the aluminum sulphate and the coloring matter has another bearing upon the coagulation of waters. Fuller found at Louisville that the theoretical amount of reduction of alkalinity by the addition of a certain amount of aluminum sulphate was seldom obtained in practice. This he attributed chiefly to an absorption of the chemical by the suspended matter in the water. It is evident from the foregoing that the coloring matter in water also

* "The Filtration of Public Water Supplies."

Mr. Whipple. absorbs a certain amount of chemical, though not in just the same manner. For example, when 1 grain per gallon was used to decolorize a water which had a color of 75, the reduction of alkalinity was only 84% of the theoretical amount. In this case the coagulant was not lost, of course, as it was doing useful work in precipitating the coloring matter; but, inasmuch as the precipitate formed by this reaction has been found by experiment to be far less efficient in coagulating the suspended matter than is aluminum hydrate, it follows that more coagulant is required to clarify a colored water than a water without color.

When a water is both colored and turbid the problem becomes a complicated one. If the water has no alkalinity the aluminum sulphate is in part absorbed by the suspended matter and in part by the dissolved organic matter. The precipitate formed in the latter case has but little coagulating effect, as shown by the following experiment: A non-alkaline water which had a color of 200 and a turbidity of 50 parts per million on the silica scale was neither clarified nor decolorized by the addition of 2.5 grains of aluminum sulphate per gallon, although the same water without turbidity was decolorized by 1.3 grains, and water of the same turbidity, but without color, was clarified by using less than 1 grain.

If a colored turbid water has a sufficient amount of turbidity the action that takes place is as follows: Part of the aluminum sulphate is absorbed by the suspended matter and is practically wasted; part unites chemically with the coloring matter and is lost as a coagulating agent, although it is doing useful work; part is decomposed by the carbonates of the water, with the formation of aluminum hydrate; of this hydrate a part unites with the coloring matter and is lost as a coagulating agent, while the remainder acts as a true coagulant, coating the suspended particles and uniting them into flocs.

As a rule, turbid waters are not high-colored, although the occurrence of turbidity and color in the same water is by no means unknown. The Hudson River at Albany at certain seasons of the year contains considerable coloring matter in addition to its turbidity, and the water from swamps is often somewhat turbid after heavy rains. The effect of turbidity upon decolorization is therefore of no little practical importance. Unfortunately, the data for measuring this effect are not at hand. Certain facts bearing upon the question, however, have been noticed. If an amount of aluminum sulphate insufficient to decolorize a water is added no effect is produced; it is necessary to add enough to cause an almost complete precipitation of the coloring matter before any action is observed. And in the case of a water which is both colored and turbid it is necessary to add enough coagulant to both clarify and decolorize before any action is observed. In other words, practically speaking, it is impossible to decolorize a

water by this method while it remains turbid, or to clarify a water while it remains colored. Enough coagulant must be added to produce both results or the chemical will be wasted. In the absence of more reliable data upon this point it may be stated, as a general rule for estimating the amount of coagulant required, that in order to decolorize a water about 0.1 grain of aluminum sulphate per gallon must be added for each 5 parts of color, and that about one-half of this is active as a coagulating agent. With turbid waters the amount of suspended matter usually determines the amount of coagulant required, but when the color is above 50 the amount of coloring matter is likely to be the controlling factor. Mr. Whipple.

In most of the experiments upon decolorization previously referred to it was observed that the water was seldom left absolutely colorless; there was usually a certain amount of residual color even after coagulation and filtration. Usually, this did not exceed 10, an amount which may be practically disregarded. When, however, the color of the raw water was more than 200, the residual color became important. For example, a water which had a color of 300 left a residual color of 50 after treatment with 6.5 grains per gallon. In some cases this residual color could be removed by a second application of aluminum sulphate, but in other cases it resisted completely the action of this coagulant. Inasmuch as the residual color usually had a marked greenish hue it may be suspected that ferrous iron was present, and that this in some way prevented the action of the aluminum. Inasmuch as these very high-colored waters are seldom met with in such quantities that they would be used as a public water supply this subject has a theoretical rather than a practical interest.

Temperature has a considerable effect upon the amount of aluminum sulphate required to decolorize a water, and upon the rapidity with which the action takes place. This is illustrated by Table No. 4.

TABLE No. 4.—Effect of Temperature upon the Decolorization of a Clear Water which had a Color of 50.

Temperature. (Fahr.)	Amount of Aluminum Sulphate required to decolorize, in grains per gallon.	Time required for precipitation.
44	1.2	18 hours.
70	1.0	6 "
98	0.8	3 "

It is a well-known fact that the viscosity of water decreases with its temperature. Warm water will pass through a filter more rapidly than cold water, and, for the same reason, particles in suspension in warm water will settle more rapidly than in cold water.

Mr. Whipple. Other substances than aluminum sulphate have been suggested for use as coagulants: for example, potash alum, ferrous sulphate, ferric sulphate, ferric hydrate prepared in various ways, etc. These appear to have no advantage over aluminum sulphate, however, in point of economy. It may be observed that the iron salts do not combine directly with the coloring matter of the water to form a precipitate, as does aluminum sulphate. The use of permanganate of potash has been suggested, but the slowness with which the action takes place is an objection to its use in most cases.

Decolorization by the Use of Chemicals Alone.—There are many chemical substances which will act on the coloring matter of water. In general, acids reduce the color, while alkalies increase it. Acids do not effect complete decolorization, however, and are of no practical use except in reducing the alkalinity of the water, as previously mentioned.

The use of ozone has been advocated, and in certain cases it may in the future find a practical application. Comparatively little, however, is known as to its efficiency or cost. The use of the hypochlorites of calcium and sodium and of peroxide of chlorine has also been suggested. These are proprietary processes which embrace also some form of rapid filtration or straining. In general, however, it may be said that the decolorization of water by the use of chemicals alone does not appear to be as promising as the use of some form of coagulant in connection with rapid filtration.

Mr. Hazen. ALLEN HAZEN, M. Am. Soc. C. E.—The decolorization of water, as Mr. Whipple says, is purely an æsthetic question. Color does not affect the wholesomeness of the water, nor, in general, its usefulness for mechanical purposes. Pure water is colorless, or, to be more exact, it has an extremely light blue color. Waters carrying certain vegetable matters in solution are discolored by them, and although the coloring matters commonly found in waters are similar to, if not identical with, the coloring matter of tea, and are absolutely harmless, most people prefer that water should not carry the visible evidence of the presence of foreign matters.

The classification of the means of removing color which Mr. Whipple has given us is interesting, but at one point at least it might be carried further with advantage. The chemical processes of decolorization might be divided into two classes, namely, those in which the coloring matter is coagulated by a precipitant and removed in an insoluble form, and those in which it is oxidized and destroyed. Of these, the first, or coagulating processes, are the only ones which have been practically successful thus far. Sometimes these processes are open to objections which would be avoided by the use of the oxidizing processes. The chemical oxidizing processes consist in the application of some powerful oxidizing agents, capable of destroying

the coloring matter. Ozone, permanganate, and a compound of chlorine and oxygen have been used. These processes, of course, are different from the natural oxidizing processes, in which the coloring matter is destroyed under the action of sunlight, etc., without chemicals. It is a curious fact that the coloring matter of water yields rather easily to chemical oxidation, although it is one of the most difficult of substances to oxidize by nitrification in a filter. The decolorization of water by these chemical oxidation processes, although as yet in an entirely experimental state, offers reasonable promise of practical results in the future. Mr. Hazen.

GEORGE A. SOPER, Assoc. M. Am. Soc. C. E.—Some remarks made by Mr. Whipple and Mr. Hazen suggest that it might be of interest to refer to one line of the inquiry made by the speaker into the means of decolorizing water. Mr. Hazen, more particularly than Mr. Whipple, emphasized the fact that oxidation plays an important part in the decolorization of water. It might be said that the presence of color, to a very large extent, is due to imperfectly oxidized organic compounds. In Nature, a good deal of oxidation takes place as the water percolates through the soil, and ground-water is found to be very generally free from color. So the principles which operate in Nature to make such waters free from color have been applied in the slow filtration of water, with the result that oftentimes a very considerable removal of color is accomplished by this process. Mr. Soper.

Some years ago, as many will remember, aeration was proposed for the removal of color because it was thought that the oxygen of the air would have a direct action upon the organic impurities, would oxidize them and so destroy them, or burn them up, as it was expressed. Later, it was found that such oxidation proceeds chiefly through the agency of bacteria, and that, beyond insuring the presence of a small amount of oxygen, there was no advantage in aerating water, so far as favoring oxidation was concerned.

Although it is well known that the principal source of color in water supplies is due to substances which have been extracted from vegetable matter, and that water containing objectionable stains are often derived from swampy or ill-drained districts in which there is a heavy growth of plants, shrubs or trees, the study of the cause of color has not progressed so far as to enable us to definitely name the chemical compounds which are responsible for the trouble. Interesting as they would be, investigations into the constitution of such matters belong to a field of chemistry which has not yet been thoroughly cultivated. Practically, all we can say with positiveness is that color is due to the extractives of woody fibers in which life has been destroyed and which are ready, or have already begun, to make progress toward oxidation. Complete oxidation is the ultimate fate of these matters in Nature. When oxidation is carried to its farthest point, coloring matters of organic origin are destroyed.

Mr. Soper. The most universal objection to colored water supplies lies in their unpleasant appearance. The shade may range in depth from a degree which is hardly perceptible to a hue which can be compared with strong tea. Dark-colored waters are uninviting and frequently repellant, especially to those unaccustomed to their use, but they are not necessarily injurious to health or objectionable for use in steam boilers. From these facts it is to be inferred that colored waters are to be objected to only on grounds of minor consequence, color ranking in importance considerably below bacterial impurity, turbidity, iron, hardness and odor. Consequently, it is doubtful if it is desirable to place a limit upon the amount of color which we should consider permissible in a public water supply. Æsthetic standards can seldom be set up with any assurance that they will be universally agreed to, and there seems to be no other reason to consider a limit necessary. Upon grounds of practical expediency, therefore, the writer thinks it injudicious to attempt to establish a mark.

In these remarks it has been assumed that the existence of color without other impurity has been the subject under consideration. Where color exists along with other qualities which tend to make a water supply objectionable, the subject becomes complicated and extends beyond the range of brief discussion. A comparatively small amount of coloring matter when accompanied with turbidity, for example, is far more objectionable than the same amount in a water otherwise clear. The reasons for this properly belong to a broader topic.

While the nature of the origin of color in water suggests that in most cases it may be much more readily prevented than cured, there are several practical ways for its elimination. Unless it is present to an excessive degree it is practicable to remove color from drinking-water supplies by natural and artificial means. It has been observed that a reduction in color may be effected in open reservoirs, provided the water is not stored at too great a depth. Experiments have been made which show that sunlight is the important cause of this reduction. The action is apparently similar to the bleaching of dampened linen when the latter is exposed to the direct rays of the sun. The bleaching of linen is considered to be due to the oxidation of the organic matter which discolors the goods, and it is not improbable that the bleaching of vegetable color in reservoirs is all due to oxidation also.

The passage of water through the ground or through slow filters has for one of its objects a practical removal of color. The amount varies with the rate of percolation and the amount of color originally present. Part of the coloring matter removed under such circumstances is in a finely divided solid condition and is extracted mechanically. The dissolved matters which make up the remainder of the color are probably oxidized through the activity of micro-organisms.

Mr. Soper.

The most effective process commonly used for the removal of color is coagulation. Alum or its equivalent is used to bring together the solid particles of coloring matter and destroy that part of the remainder which is in solution. A serious objection to coagulation, however, lies in the necessity for removing from the water the voluminous precipitate which results from the use of coagulating chemicals. Settling basins or filters, or both, have to be used to accomplish this purpose, and such structures generally require an extra pumping of the whole supply. When it is necessary to add lime or other alkali to bring about a proper action of the alum, this process of color removal is open to further objection.

As is well known, charcoal, coke and analogous substances have the power of absorbing a large amount of color. Sugar refineries use charcoal filters in removing brown stains from sugar juices. Experience, up to the present time, however, indicates that the use of such materials is not practicable upon the scale required for municipal water-works, owing to the necessity of renewing or revivifying the absorbent. Consequently, beyond the introduction of filters designed to purify a few thousand gallons of water per day, charcoal and other absorbing substances have not been satisfactorily used in water purification.

It might seem that the use of peroxide of chlorine, bromine, permanganate of potash, or ozone, all of which have been recommended recently for the sterilization of drinking-water, might be advantageously used to remove color. All these chemicals are powerful oxidizing agents, and would, without doubt, strongly attack the organic substances which give water the appearance of color. It will be sufficient to describe the method of applying ozone, since many considerations make this method the most suitable of the chemical processes for the removal of color by oxidation.

The purification of water by ozone has for its object the direct and immediate destruction of organic impurities by a combination of their elements with oxygen. It was formerly supposed that the simple aeration of water, by bringing the oxygen of the atmosphere into contact with the impurities, tended to destroy them. On this principle, fountains, jets and falls were arranged at many water-works in order to aerate the waters in a thorough manner. It was found, however, that atmospheric oxygen is too stable and inert to unite rapidly with organic matters. Without the assistance of sunlight or some life process, such as that of bacteria, for example, it is doubted by many whether such combination can take place. Under the most favorable circumstances the natural oxidation of organic matter is a very slow process.

The purification of water by ozone properly lays claim to the advantages which were formerly thought to belong to aeration. By this treatment, a portion of the oxygen of the air is converted into an

Mr. Soper. active condition which enables it to unite with organic matter immediately and without the aid of bacteria or sunlight. It is consequently possible, by bringing water containing not only color, but odor and micro-organisms, into contact with ozone, to destroy these impurities and mineralize them.

There are many features which distinguish sharply the ozone method from other chemical processes of oxidation recommended for the purification of water supplies. Ozone is not soluble in water, and so cannot by any chance have an injurious effect upon those persons who may consume the latter. If an over-dose were accidently applied to the water, the residue would escape into the air and be wasted; there is no residue. Ozone is oxygen, and only oxygen, and becomes atmospheric oxygen if not used immediately after it is produced.

The apparatus for its application is less elaborate and expensive than that required for filtration. Extra pumping is not necessary. There is no cleaning to be done. Whatever cost is involved in the construction of an ozone plant is chiefly confined to the machinery required to ozonize the air. The cost of operation is chiefly the cost of a few horse-powers of electric energy. From 4 to 5 grams of ozone, O_3, may be obtained per cubic meter of air. From 30 to 50 grams may be produced per horse-power of energy. For sterilization, water requires from 6 to 10 parts of ozone per million parts of water, or in other words, about 24 kgrs., or 53 lbs., of ozone per million gallons.

To any one who may doubt the practicability of producing ozone on a large scale, it may be said that the total capacity of works operated by a manufacturing chemical company of which the speaker has knowledge, is fully ½ ton of ozone per day. This should be sufficient to sterilize 20 000 000 galls. per twenty-four hours.

In conclusion, the writer considers that the main principles upon which depends the successful use of ozone in purifying water are fairly clear. The sanitary value of ozonization has been declared in the official publication of the Pasteur Institute of France and the Imperial Board of Health of Germany. The International Congress of Hygiene and Demography, which met at Paris last year, discussed the subject favorably. Descriptive papers, read before engineering and water-works associations in several countries, have recommended ozone. With the hygienic value of the process thus fairly proved, the question is to apply ozonization in purifying water so that it may be economical. This is an engineering proposition, and one which is not likely to wait long for a practical application in America.

Mr. Fuller. GEORGE W. FULLER, Assoc. M. Am. Soc. C. E.—The discussions of the preceding speakers seem to cover the leading information now available upon this topic in a very comprehensive manner, and leave very little which can be added in general terms.

As is well known, this subject is quite new, comparatively speak-

ing. There is one phase of it to which attention might be called. Mr. Fuller.
That is, the practical significance of the subject at this time is not normally related to those water supplies which are satisfactory in all ways except an occasional moderate excess of color. It deals with the consideration which is to be given to the matter in connection with the development of new supplies, and the general improvement and extension of existing works, so that the result will meet satisfactorily the criticisms of the future.

This question is very largely an æsthetic one. There is no doubt that the average water consumer is better satisfied with a colorless than with a colored water.. In the years to come, it seems certain that the preference will become sufficiently pronounced in many cases, so that citizens will insist upon the decolorization of their water supplies, a process which ordinarily involves only a very small cost, when expressed as an annual *per capita* charge.

The general views which have been expressed, along this line and also that of methods of color removal, by the gentlemen who have preceded the speaker, coincide very closely with his own. There is only one point, among those which have been mentioned specifically, on which he has any comments to add at this time. That refers to the amount of chemical treatment which is necessary for the removal of a given quantity of color, in the instance of those colored waters which cannot in a practicable manner be rendered satisfactory in appearance by the elimination of swamps upon the water-shed, or by the bleaching action that is found in large reservoirs.

As a general proposition, it may be said that the amount of sulphate of alumina, when chemical decolorization is used, which must be added to a water, to effect a substantially complete removal of the color, is approximately in the ratio of 2 to 2.5 grains per gallon for a color of 100, according to the platinum standard, as stated by Mr. Whipple. Casual observations along this line have been made from time to time upon quite a number of waters coming from various sources throughout the country. There is very little of this information, however, which has been obtained on a large scale up to this time, the purification plant at Norfolk, Va., being, the speaker believes, the only one where such records are systematically secured. While in a majority of cases the waters studied have given results which are within reasonable accord with the above-stated rule, it is to be noted that some waters seem to form exceptions in this respect.

In connection with some investigations which are being made at New Orleans, in comparing various local waters there, for adaptation to a new supply, some attention has been given to the color of the water obtained from deep wells. These wells are some 600 to 800 ft. deep. The waters contain very little suspended matter, as a rule, there being apparently some 5 to 10 parts per million of suspended

Mr. Fuller. organic matter. In color, the waters are high, the amount ranging ordinarily from about 150 to 350, platinum scale. The alkalinity is also high, due to alkaline carbonates and not to lime and magnesia.

In conjunction with Mr. R. S. Weston, the resident expert in charge of the New Orleans investigations, the speaker has given some study to the amount of sulphate of alumina necessary for the decolorization of the local well-water. The leading results of these studies are summarized in Table No. 5.

TABLE No. 5.—RESULTS OF EXPERIMENTS UPON THE DECOLORIZATION OF HIGHLY COLORED WATERS FROM DEEP WELLS AT NEW ORLEANS.

Sample.	PARTS PER MILLION.		GRAINS PER GALLON OF SULPHATE OF ALUMINA.		
	Alkalinity.	Color, by Platinum Standard.	To decolorize.		To effect a beginning in color removal from original water.
			Original water.	Water after neutralization.	
A	412	190	13	3.0	10
B	365	200	5	0.5	3
C	420	120	7	1.2	5
D	403	350	7	2.5	5
E	441	180	18	3.0	6

A detailed comparison of these data with those presented by Mr. Whipple, may, perhaps, bring out some points of interest. In a general way, it may be said that they do not seem to be at variance with his explanations and theories. The New Orleans well-waters appear to have in some cases an individuality of their own, especially when compared with surface-waters, in connection with the treatment in question. How far this is due to the composition of the organic matter, to the mineral contents of the water, or to a combination of the mineral and organic matters, is a point which cannot now be settled. Whatever the relative significance of the several factors may be, it is of some practical importance to know that some waters, in proportion to their color, are decolorized with much more difficulty than others.

Mr. Maignen. J. P. A. MAIGNEN, Assoc. Am. Soc. C. E.—In order to judge whether it is necessary to decolorize water there should be a clear conception of what it is that gives the color.

It is assumed that the class of water now under consideration is that which flows in the rivers of the Eastern States and in those along the Atlantic Coast of this country, and which is also of the same class as that found in Russia, Germany, Holland, Scotland, Ireland and many other places on both continents.

A good type of this water flows in the Black River at Watertown. In a glass it is perfectly transparent, but it looks like bright light tea.

It has been said that this color is in solution, because neither filter

paper nor sand nor porcelain filters can get rid of it. This demonstration does not prove that the color is not due to molecules in suspension. It simply proves that neither paper nor sand nor porcelain filters are fine enough to retain them. Mr. Maignen.

The Neva water at St. Petersburg in the summer, the Seine water at Paris in the autumn, the peaty waters of New York State and many other places, show that what gives color and odor to such waters is not some unknown liquid substance, but that it is due to clearly defined molecules of vegetable and animal matter in decomposition, capable of being seen with the microscope and isolated by proper filtration.

In many places the natives attribute to these waters fevers and dysentery troubles and shun their use as a public danger, preferring to drink well-waters, which are often worse but which do not give ocular evidence of contamination. It is therefore concluded that all colored waters ought to be decolorized.

Decolorization has been accomplished lately in this country by coagulation and filtration, but this system is expensive and unsatisfactory.

There are two filtering materials which are capable of removing the color of peaty waters without the use of coagulants. They are asbestos and charcoal.

Asbestos is efficient because the fibers, and therefore the voids between them, are indefinitely finer than those of filtering paper, sand or porcelain filters. By filtration through these substances all color and odor can be removed. This is not due to some chemical or physico-chemical action, but to purely physical action or fine filtration.

The qualities of charcoal as an absorbent of gases and color have long been known. Nothing has been found capable of replacing it in sugar refining. The speaker has found that it will remove the color and odor of peaty waters as well as organic matter in solution and dissolved metallic salts, such as lead, iron, copper, etc.

This quality has been sometimes attributed to a physico-chemical action and sometimes to molecular attraction. This point will not be discussed here. Suffice it to say that in sand and porcelain filters the units that make up the filters are solid or non-porous. In charcoal, on the contrary, each unit is honeycombed by an infinite number of microscopic channels. While in the former materials the filtration takes place in the voids or meshes, in the latter it goes on through the whole mass, through each unit as well as between the units. What goes on is more of the nature of dialysis or osmotic action. In other words, it is an extremely fine kind of filtration which is much more intense than through non-porous material.

To the experts in filtration in this country belongs the task of finding out how the charcoal should be used. The speaker obtains the best results by depositing a layer of finely powdered charcoal on asbestos cloth; but there are many other ways of using charcoal for

Mr. Maignen

the purpose of decolorizing water. It will certainly be found that there is more purifying power in one ounce of charcoal than in many ounces of other filtering materials.

Objections to charcoal have been raised in London, and an echo of these objections is contained in Mr. Whipple's discussion, but the speaker believes that these objections are not in the slightest degree justified by careful inquiry.

It has been stated, first in London, that "charcoal is favorable to the growth of micro-organisms." This statement has been based upon experiments made by Dr. Percy Frankland and recorded in his book* under the heading "Efficiency of Different Filtering Materials." There may be found the following table:

Filtering material.		Micro-Organisms per Cubic Centimeter.		Reduction per cent.	Approximate rate of filtration per square foot per hour.
		Unfiltered water.	Filtered water.		
Iron sponge.	1st experiment.....	80	0	100.0	....
	2d experiment, after 12 days' action.....	2 800	0	100.0	0.40
	3d experiment, after 1 month's action.....	1 280	2	99.8	0.45
Animal charcoal.	1st experiment.....	Too numerous to count.	0	100.0	....
	2d experiment, after 12 days' action.....	2 800	0	100.0	0.46
	3d experiment, after 1 month's action.....	1 280	7 000	increase 447.0	0.86

It will be observed that in the second and third experiments with both iron sponge and animal charcoal, Mr. Frankland applied the same water, but he did not do so in the first experiment.

The water applied to the iron sponge filter contained only 80 bacteria, while that applied to the animal charcoal filter contained bacteria "too numerous to count." This ought to have thrown the whole experiment out of consideration.

Another fact will be observed from this table. Where he found the increase in the number of bacteria, the speed of filtration was doubled, being 0.86 gall. per square foot per hour instead of 0.45 or 0.46 in the other experiments. This surely ought to have made an impression on Dr. Frankland. The filter must certainly have been broken or otherwise damaged, and he ought to have begun his experiment over again.

Other experiments have since been made, but, like those of Dr.

* "Micro-organisms in Water," 1885.

Frankland, they have not been carried on long enough to be of any real value; on the contrary, they serve to confirm the original error, as will be shown. Mr. Maignen.

Table No. 6 is a record of a series of experiments to ascertain the exact efficiency of charcoal in removing bacteria. Three small filters were installed, each containing the same quantity and the same charcoal (*Carbo-Calcis*), so as to have a triple check. The precaution was taken to use during the first month of the experiment filtered water which was practically sterile, containing generally less than 40 bacteria. During the first three or four weeks the effluent from each of the three filters had a very large number of bacteria. These bacteria did not come from the water, as the water applied contained so few, but from the charcoal. How did they get in the charcoal? Simply from the air which entered the pores on cooling after burning.

Readers of Tyndall's book, "Floating Matter in the Air," will remember his most interesting statement concerning grown-up bacteria and dry spores. He showed that it required a certain period of incubation in water for the growth and development of the spores, and he found that it was necessary to boil the water three times at intervals in order to get rid of all the germs, as they could not be destroyed by a single boiling.

During the first period of use of charcoal filters the dry spores which fill its pores gradually grow into bacteria and are carried out by the filtered water. It must be remembered that the charcoal contains at least fifteen times its volume of atmospheric air, and therefore, a considerable quantity of dry dust with it; hence it is that water going in with 16 or 18 bacteria comes out with 3 000 or 4 000.

In these experiments it will be seen that it has taken fully one month of daily use to free the charcoal of what may be called "constitutional" germs or bacteria. Anyone starting such an experiment and stopping it at the end of a week, a fortnight or three weeks, would be justified in condemning charcoal, but if he persevered beyond the month he would find all the excellent qualities of charcoal as a bacterial filtering material.

As soon as the filter was found to be ripe, or deprived of its constitutional bacteria, on October 17th, raw water with many bacteria began to be introduced. The filtrate of the three filters remained practically sterile.

From September 10th to December 15th the supply of the filter was intermittent. Water was put in once a day. Then the operation was suspended until December 24th and the filters were placed under running water from the tap so that they filtered day and night. It will be seen that the results remained thoroughly satisfactory. These results will show the advantage of perseverance and the danger of basing conclusions on insufficient data.

Mr. Maignen.

TABLE No. 6.—Record of Experiments made to Ascertain the Bacterial Efficiency of Charcoal Filters.

Dates.		Water Used.		Water Filtered Through *Carbo-Calcis*.					
				Filter No. 1.		Filter No. 2.		Filter No. 3.	
Sampling.	Examination.	Total number of bacteria per cubic centimeter.	Liquefying.	Total number of bacteria per cubic centimeter.	Liquefying.	Total number of bacteria per cubic centimeter.	Liquefying.	Total number of bacteria per cubic centimeter.	Liquefying.
	Sept. 13	36	8	412	12	300	24	620	16
Sept. 10	15	32	8	360	10	276	18	520	16
11	15	28	8	316	8	240	14	460	20
12	16	0	0	3 720	30	3 360	20	3 420	24
13	16	16	0	4 140	10	4 224	8	3 700	0
14	19	18	6	5 600	0	9 900	0	6 120	16
15	20	16	4	4 800	0	7 740	0	5 720	0
17	22	22	4	4 500	0	6 480	0	5 340	0
19	22	16	6	4 160	0	5 940	0	4 640	0
20	23	28	10	3 920	0	4 860	0	4 180	0
21	26	34	12	3 064	0	4 388	0	3 358	0
22	27	16	4	2 678	0	3 346	0	2 534	0
24	28	20	4	2 326	10	2 634	8	2 196	14
25	29	14	0	2 032	6	2 216	0	1 968	16
26	29	120	44	1 800	0	2 098	12	1 920	16
27	Oct. 2	84	16	1 660	0	2 430	30	1 808	18
28	2	36	18	1 560	0	2 180	0	1 620	2
29	3	16	0	1 500	0	2 032	6	1 520	0
Oct.	1900								
1	4	16	4	1 250	0	1 460	0	1 280	6
3	6	18	0	1 060	0	1 200	0	1 040	0
4	10	18	4	980	0	1 120	0	990	0
5	10	1 048	46	926	12	1 076	8	910	2
6	9	16	0	624	0	1 060	16	740	0
8	11	16	0	60	0	180	0	36	0
9	13	26	0	40	0	190	0	16	0
10	14	28	0	16	0	80	0	14	0
11	15	26	0	10	0	18	0	14	0
12	16	8	0	4	0	2	0	8	0
13	17	10	0	0	0	0	0	4	0
15	19	8	0	0	0	0	0	4	0
16	20	10	0	0	0	0	0	2	0
17	20	980	90	6	0	0	0	6	0
18	22	740	80	4	0	4	0	6	0
19	23	850	60	6	0	4	0	8	0
20	24	1 600	120	4	0	4	0	6	0
22	25	1 360	80	6	0	6	0	4	0
23	26	1 260	130	4	0	4	0	4	0
24	27	2 500	120	0	0	0	0	4	0
25	29	2 200	120	2	0	8	0	0	0
26	31	1 650	80	6	0	8	0	2	0
27	Nov. 1	1 250	90	4	0	6	0	4	0
29	2	1 420	100	10	0	12	0	6	0
31	5	1 340	100	8	0	0	0	4	0
Nov.	1900								
1	5	1 650	140	4	0	6	0	4	0
2	6	1 900	150	0	0	6	0	8	0
3	8	2 150	180	4	0	0	0	4	0
5	8	1 720	130	0	0	6	0	2	0
6	9	3 220	220	4	0	0	0	2	0
7	10	3 840	190	6	0	2	0	2	0
8	12	2 000	160	1	0	8	0	2	0
9	13	2 900	260	7	0	6	0	11	0
10	14	4 350	200	4	0	0	0	3	0

TABLE No. 6—(*Continued*). Mr. Maignen.

Dates.		Water Used.		Water Filtered Through *Carbo-Calcis*.					
				Filter No. 1.		Filter No. 2.		Filter No. 3.	
Sampling.	Examination.	Total number of bacteria per cubic centimeter.	Liquefying.	Total number of bacteria per cubic centimeter.	Liquefying.	Total number of bacteria per cubic centimeter.	Liquefying.	Total number of bacteria per cubic centimeter.	Liquefying.
Nov. 12	Nov. 16	2 700	160	8	0	7	0	0	0
13	17	1 920	140	3	0	0	0	5	0
14	17	1 600	100	0	0	2	0	0	0
15	19	2 090	160	8	0	6	0	4	0
16	20	2 630	200	8	0	0	0	4	0
17	21	5 110	270	6	0	8	0	12	0
19	23	2 700	210	4	0	10	0	7	0
20	24	3 200	130	12	0	6	0	12	0
21	24	5 250	400	2	0	10	0	6	0
22	26	2 460	230	6	0	4	0	2	0
23	27	18 600	800	10	0	15	0	12	0
24	28	6 400	500	8	0	6	0	11	0
26	30	5 000	230	7	0	11	0	5	0
27	Dec. 1	5 820	320	4	0	5	0	2	0
28	3	8 150	400	3	0	9	0	1	0
30	4	13 000	590	6	0	5	0	2	0
Dec.	1900.								
1	5	11 560	500	9	0	7	0	4	0
3	7	11 000	410	4	0	6	0	7	0
4	8	5 640	160	3	0	2	0	6	0
5	8	6 950	200	3	0	4	0	1	0
7	11	9 760	190	6	0	7	0	4	0
8	12	12 100	210	2	0	8	0	1	0
10	14	10 620	170	3	0	4	0	8	0
11	15	9 340	140	6	0	2	0	3	0
12	17	13 200	220	7	0	0	0	2	0
13	18	16 240	340	1	0	5	0	3	0
14	18	10 430	260	0	0	8	0	11	0
15	19	9 210	160	6	0	8	0	2	0
24	29	11 500	200	6	0	10	0	12	0
27	31	9 700	150	4	0	0	0	8	0
28	Jan. 1	9 900	210	3	0	6	0	8	0
29	3	11 400	280	1	0	3	0	5	0
Jan.	1901.								
1	8	13 400	320	2	0	1	0	3	0
4	9	9 620	160	2	0	3	0	4	0
5	10	10 800	210	0	0	4	0	6	0
7	11	13 090	230	2	0	4	0	1	0
8	12	11 100	210	4	0	2	0	6	0
9	14	10 060	260	0	0	8	0	9	0
10	15	14 090	300	5	0	2	0	10	0
11	16	11 520	160	9	0	0	0	6	0
12	17	12 420	180	4	0	2	0	3	0
14	19	9 820	150	10	0	0	0	9	0
15	19	9 230	120	6	0	3	0	8	0
16	21	16 750	500	2	0	0	0	0	0
17	22	28 500	1 260	16	0	18	0	11	0
18	23	20 100	720	12	0	8	0	2	0
19	24	13 950	630	9	0	9	0	13	0
22	26	9 520	110	12	0	9	0	2	0
23	28	12 180	610	10	0	12	0	1	0
24	29	11 210	610	4	0	6	0	3	0
25	29	6 950	590	22	0	25	0	20	0
26	31	10 140	460	17	0	13	0	9	0
28	Feb. 2	10 750	500	9	0	14	0	0	0
29	3	13 060	610	9	0	10	0	2	0

Mr. Maignen.

TABLE No. 6—(*Concluded*).

Dates.		Water Used.		Water Filtered Through *Carbo-Calcis*.					
				Filter No. 1.		Filter No. 2.		Filter No. 3.	
Sampling.	Examination.	Total number of bacteria per cubic centimeter.	Liquefying.	Total number of bacteria per cubic centimeter.	Liquefying.	Total number of bacteria per cubic centimeter.	Liquefying.	Total number of bacteria per cubic centimeter.	Liquefying.
Jan. 30	Feb. 4	11 500	530	5	0	6	0	3	0
31	5	9 930	450	8	0	10	0	0	0
Feb. 1901.									
1	5	10 060	490	4	0	7	0	4	0
5	9	12 030	490	3	0	5	0	1	0
6	11	12 270	500	0	0	4	0	1	0
7	12	14 630	330	3	0	6	0	6	0
8	12	11 930	190	0	0	1	0	8	0
9	14	8 260	120	3	0	2	0	8	0
11	15	9 030	110	0	0	1	0	4	0
12	16	11 430	180	1	0	6	0	2	0
13	18	9 980	160	1	0	1	0	0	0
14	18	8 780	150	0	0	0	0	6	0
15	19	8 120	130	1	0	2	0	1	0
16	21	15 980	180	6	0	7	0	2	0
18	22	15 360	200	1	0	3	0	0	0
19	23	12 540	200	4	0	1	0	0	0
20	25	10 710	130	6	0	0	0	3	0
21	25	14 270	250	4	0	1	0	4	0
22	27	9 940	190	0	0	0	0	3	0
23	27	10 980	200	0	0	1	0	1	0
25	Mar. 1	12 830	230	6	0	1	0	2	0
26	3	8 160	160	2	0	5	0	3	0
27	3	8 220	210	3	0	7	0	8	0

Mr. Chester.

J. N. Chester, M. Am. Soc. C. E.—Being in the position of representing, in most part, private, or quasi-public, corporations delivering water to municipalities, the speaker is probably placed under a ban more strict than that over the municipality which furnishes water to itself, or of an engineer for a corporation furnishing water to itself, and can only emphasize the necessity, from his own standpoint, of removing all color. Color gives just as much trouble as turbidity, and the municipality buying water from a private corporation will not, if it can help it, tolerate either. As to the methods of removing color, one method will not fit in all cases, and, while the theory advanced by Mr. Maignen is that color may not be in solution, but absolutely in suspension, and is one which had not occurred to the speaker before, yet the fact that such color is in suspension has always been proved by the absence of any precipitate after allowing the water to stand for days and weeks in an absolutely clear vessel. Nevertheless, it has been possible to remove color in suspension, except that caused by tan bark or tannin, in water taken from the fore or back bay where logs were lying, or from the river in which

there are periodical log drives. It has been found that most of that color can be coagulated and removed by the use of chemicals, while in other waters the very same chemicals, in any proportion, failed to remove the color. Mr. Chester.

FREDERIC P. STEARNS, M. Am. Soc. C. E.—Mr. Whipple, in opening the discussion upon this topic, has covered the ground very thoroughly, so that the speaker can add nothing of interest to the general discussion of methods, and will therefore confine his remarks to work now in progress, relating to the decolorization of the water supply of the Metropolitan Water District of Massachusetts, which includes the City of Boston and seventeen other cities and towns. Mr. Stearns.

The streams formerly used as the main sources of supply of the City of Boston derived a sufficient proportion of their water from swamps to give it a brownish tinge, and the average color of the water in the streams was about 90, on the scale used by Mr. Whipple. A part—approximately one-third—of this color was removed by storage in the reservoirs.

In selecting a new source of supply for the Metropolitan District, care was taken, in the first instance, to select a drainage area in a hilly country where there was less swampy land and where the water had naturally only about one-half the color of the streams supplying the City of Boston. Notwithstanding this precaution, two measures were adopted for decolorizing the water:

1. The construction of an extremely large storage reservoir, from which all soil and vegetable matter are to be removed, in which the water will remain long enough to bleach.

2. The drainage of swamps by drainage ditches located near their borders, to intercept the upland water and prevent it from entering the swamps.

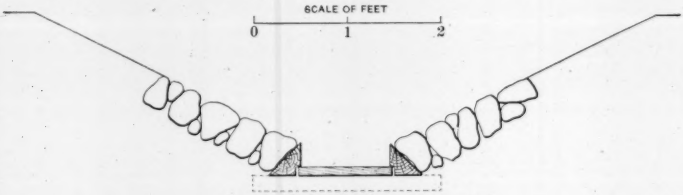
The Wachusett Reservoir, now under construction for the storage and bleaching of the water, will hold an amount of water equal to 533 000 000 galls. per square mile of drainage area. This is far in excess of the usual practice; for instance, the storage of the City of Boston on its principal water-sheds, in 1897, was equivalent to 90 000 000 galls. per square mile of water-shed, or only about one-sixth of the amount to be furnished on the Wachusett water-shed. To express the storage in the Wachusett Reservoir in another way, it may be said that it is equivalent to the average flow of the streams for $1\frac{1}{3}$ years.

The drainage of swamps for the purpose of diminishing the amount of color and organic matter in water is somewhat novel, and it may be of interest to refer in some detail to the work which has been done in this line up to the present time.

On a portion of the Sudbury water-shed, having an area of 27.68 sq. miles, which is used in connection with the new water-shed for

Mr. Stearns. supplying the greater portion of the water, swamps having a total area of 937 acres have been drained by the construction of 129 510 ft.—equal to 24.53 miles—of ditches. One of the swamps, known as Crane Swamp, is nearly as large as all the others, having an area of 460 acres, and this is drained by 45 250 ft. of ditches. The water-shed of the swamp, including the swamp area, is 2.9 sq. miles.

On account of the extent of the work, it was very desirable to adopt a form of ditch which would be inexpensive, maintain its form, be easily cleaned, and concentrate the flow so that leaves and other light substances would be carried along by the current. The form adopted is shown in Fig. 6.



SECTION OF SWAMP DRAINAGE DITCH

FIG. 6.

There is a wooden bottom made of a hemlock board 12 ins. wide and 1 in. thick, laid lengthwise of the ditch, and temporarily secured by nailing to 2 x 4-in. sills, 2 ft. long. At each side of the hemlock board is placed a triangular strip, formed by sawing a 4 x 4-in. spruce joist on one of its diagonals. These are rabbeted at the right-angled corner, so that they lap about $\frac{3}{4}$-in. on the hemlock board, and are expected to hold it in place after the nails have rusted away. The inclined sides of these triangular strips furnish a convenient footing for light paving, which is carried up the sides of the ditch as far as the ordinary water level in the spring of the year. Above this the slopes of earth are grassed.

All ditches, of whatever depth, have the same cross-section at the bottom, and it is a very easy matter to clean them. Near the mouths of the more important ditches large settling basins have been constructed, to intercept leaves and other substances carried along the ditches by the current. As these ditches are all constructed below the swamp level, it is expected that the bottom board and the greater portion of the strips at the sides will be covered with water at all times, so that the wood will not decay; but they have not yet stood the test of time. They seem to be very efficient.

Including work done upon the Wachusett water-shed, 1 069 acres of swamp have been drained by the construction of 144 558 ft. of ditches. The cost, including the purchase of a portion of the land,

Mr. Stearns.

the lowering and rebuilding of several road culverts, and the engineering expenses, not only in connection with swamps already drained but, in addition, for many surveys of swamps yet remaining to be drained, and which cannot well be separated, amounts to $93 per acre of swamp and 69 cents per foot of ditch. The cost of labor and materials for constructing the ditches, exclusive of land and engineering expenses, has varied considerably under different conditions, but in some swamps has been as low as 23 cents per foot.

The extent of the decolorization is indicated by Table No. 7, which gives the result at Crane Swamp and at one other swamp.

TABLE No. 7.—Color of Water from Crane Swamp and from Swamp Southwest of Marlborough Junction Before and After Draining.

Month.	Crane Swamp.		Swamp Southwest of Marlborough Junction.	
	Before draining. 1894–95.	After draining. 1900.	Before draining. 1894–95.	After draining. 1900.
January	190	90	122	55
February	220	140	165	75
March	140	75	114	35
April	154	90	128	60
May	340	40	228	40
June	248	65	200	45
July	195	47	128	40
August	315	30	200	35
September	179	*	148	*
October	268	65	90	50
November	230	105	98	25
December	200	155	114	55
Average, omitting September	227	82	144	47

* No water running in brook.

The arithmetical mean of the results for eleven months, as shown by the table, indicates the removal of 64% of the color from the water issuing from Crane Swamp and of 67% of that issuing from the other swamp. These results, however, may be somewhat misleading, because they give equal weight to observations made in the dry and wet months. Moreover, they represent only the comparison of two years, and it is well known that in different years the color of water varies very much with the amount and distribution of the rainfall. After giving due weight to the quantity of water flowing in the different months, and after making a comparison of the color during the years included in the table with the average color for a series of years, it seems probable that rather more than 50% of the color of the water flowing from these swamps has been removed by the ditching.

In other swamps which occupy a smaller proportion of the water-

Mr. Stearns. shed of the streams in which they are located, so that more water passes through them, the original color was lower and the percentage of color removed was less; but in such cases the improvement affects a larger quantity of water in proportion to the area of swamp drained.

It is believed that the two methods adopted, namely, long storage in a clean reservoir and the efficient drainage of the swamps, will result in a practically colorless water for the supply of the Metropolitan District.

Mr. FitzGerald. DESMOND FITZGERALD, Past-President, Am. Soc. C. E.—The decolorization of water by the use of chemicals and by filtration in contact with various substances, such as animal charcoal, has received a great deal of attention, but there are comparatively few accurate data available in connection with the decolorization of water by exposure to sunlight, and by draining the swamps, which are the principal source of color.

About ten years ago the speaker began taking observations on the colors of brooks in connection with plans for improving the sources of supply of the City of Boston. Experiments and observations were made as to the source of color of surface-waters, and, at the same time, a method was devised for reading the colors accurately. The results of these investigations were published largely in the reports of the Boston Water Board. Mr. Hazen's discovery of the Platinum-Cobalt Standard, used with an accurate colorimeter,* has opened a way for the taking of colors accurately. On the Platinum-Cobalt scale the divisions are of equal length. That is to say, a color of 30 is three times that of 10. The Nessler scale, on the other hand, is an inaccurate one, as may be seen by the following comparison:

Comparison between the Platinum and Natural-Water (or Nessler) Scales.

Platinum.....	10	20	30	40	50	60	70	80	90	100
Nessler.....	0.06	0.13	0.26	0.42	0.57	0.74	0.90	0.99	1.13	1.27

The color of waters in New England is generally derived from vegetable matter, although in some cases it is dependent upon iron and manganese combined with the extractive organic matter.

In Lake Cochituate, during the summer, the water drawn from the bottom is always darker than that at the surface, and it increases in color rapidly after exposure to the air. The average of five different determinations, made between the last of August and the beginning of November, when the water at the bottom is at its maximum color, during three successive seasons, showed that the water in the bottom layers had a color on collection equal to six times that at the surface, and on exposure the color increased to twenty times that at the surface. A large part of this color is due to the presence of iron.

* Such as that described in the *Journal* of the Franklin Institute, December, 1894.

Mr. FitzGerald.

While exposed at the surface the iron is oxidized to a precipitate of ferric hydrate, which falls to the bottom, where it gives up its oxygen for the further oxidation of organic matter which is there undergoing decomposition. The iron is thus nearly all reduced to a ferrous or colorless form, which, by its oxidation on exposure after collection, causes the increase of color due to its presence in the ferric condition. In contact with organic matter this ferric iron remains soluble for a time, but finally settles out in ferric hydrate, leaving the water colorless.

It will thus be seen that iron may aid in the oxidation or purification of the water at the bottom of a pond, by giving up its oxygen. Where there is insufficient organic matter in the water to reduce it all to the soluble condition, "pond ore" will be formed in the bottom of the pond. In 1892 an elaborate investigation of this phenomenon was made, under the speaker's direction, by Mr. Frederick S. Hollis, and his full report was published in the seventeenth annual report of the Boston Water Board. Disregarding the effects due to iron, the color of surface-waters in New England is principally due to the carbonaceous matter derived from the swamps. The water of the brooks is generally a clear brownish-yellow, and represents a mean between the extremes of the greenish-yellow color imparted by freshly decomposing organic matter and the dark brown of the older peaty materials. In all investigations in connection with the decolorization of water it should be remembered that besides taking the colors of the waters as they flow in the brooks, these colors should be weighted by the amount of water flowing.

It is well known that water when exposed to sunlight for a sufficient length of time will lose all its color. This accounts for the colorless condition of the water in almost all large lakes, notwithstanding the fact that the streams supplying the lakes may be highly colored. A careful determination of the effect on the storage in a large reservoir filled in the spring and drawn down in the summer has shown that in ordinary practice about 33% of the color is destroyed.

In 1894 some experiments were made under the speaker's direction to determine the effects of sunlight; and the results, with tables, were published in the nineteenth annual report of the Boston Water Board. The following are some of the conclusions arrived at: When water is kept entirely in the dark its color remains unchanged. The nitrogen as albuminoid and free ammonia is reduced, and the nitrites and nitrates increased in the dark, while in the sunlight the albuminoid and free ammonia and nitrites are increased and the nitrates are reduced. It is found that while the water is bleached more in the summer than in the winter in ponds and lakes, the reduction per 100 hours of sunlight is practically constant for all seasons of the year. The reduction in color varies almost directly with the amount

Mr. FitzGerald. of color present, being in each case about 20% of the original color. It takes about 500 hours' exposure of bright sunlight to remove all the color, but it must be remembered that this applies to water entirely exposed in large bottles. It has generally been found in practice that water should be stored in a reservoir for nearly two years to be decolorized. During the winter, of course, when the surface is frozen over, there is little decolorization of the water by sunlight.

Although a reduction in color is accompanied by an increase in temperature, it has been found that heat alone will not reduce the color; neither does sedimentation aid in the process, at least in the case of Boston waters. To determine this fact samples of water were exposed in glass jars having discs of glass arranged in a series of horizontal layers about $\frac{1}{2}$ in. apart. Samples were also exposed in glass tubes 4 ft. long.

It does not seem probable that the bacteria can have any influence in the reduction of color, for the reason that organisms and bacteria died out in the samples exposed to the sunlight as well as in those kept in the dark. In the case of water kept in bottles of different colored glass, it was found that the color was reduced most in the white and blue bottles, being about the same in both cases, while in the red and yellow bottles the reduction of color was only about half as great, showing that the blue or actinic rays are the most efficacious.

Mr. Dunham. H. F. DUNHAM, M. Am. Soc. C. E.—It appears from this discussion that the decolorization of water is almost exclusively a bacterial process, and it also appears that it may be effected perfectly by the ozone process. The two processes have apparently little in common, and the subject invites further consideration. Slow sand filtration might be regarded as largely a bacterial process, as it is usually introduced where water contains large numbers of bacteria, and during some stage of the process of filtration the numbers are greatly increased; further, they make war upon each other until the effluent is not only sterile or very nearly so, but in such condition, with respect to substances upon which bacteria can exist, that it is expected in almost every instance that the number of those remaining will decrease rather than increase. It would appear, then, that if bacteria could decolorize the water, the ordinary sand filter affords an excellent opportunity to secure such beneficial action. But, as a matter of fact, the color and vegetable matter are not usually removed to so great an extent by the use of a sand filter as they are by some of the processes incident to mechanical filtration. It is evident that the removal of color is accomplished perfectly in the changes that water undergoes to become spring-water. It is true that a large part of the spring-water throughout the country has its direct source in the rainfall, but

it is also true, as proved by observations upon many streams during times of high water, that large numbers of bacteria are washed into the streams from the soil, and the rain-water after coming from the clouds must in many instances be not only charged with sediment from the ground and with color from decaying leaves and vegetable matter, but it also has a fair complement of bacteria before it finds its way into the earth. Mr. Dunham.

It is known that few bacteria exist below a depth of 2 or 3 ft., which is not greater than the depth of material in many of the sand filters commonly in use. There are reasons for believing that the water loses a considerable amount of vegetable matter as it passes through soil which does not contain bacteria in any considerable numbers. Evidence or proof of this may be cited in the quality of spring-water and in the microscopical examination of fine material taken at depths of many feet in the earth, as in mines. This material has been found to consist of extremely minute particles of vegetable matter which have been identified with vegetable substances existing upon the surface. Without making use of this as an argument to indicate the degree of improvement, it is possible that such considerations might lead to the suggestion that a greater depth of filter bed, which would not add greatly to the cost of a filter plant, might be of such value as to warrant its introduction where the raw water was very highly colored.

The change that water undergoes when in a large body, as in one of our great lakes, may be more like an ozone than a bacterial process. In going out from shore the number of bacteria usually diminishes very rapidly, and at a comparatively short distance a number, generally recognized as normal, may be expected, but from facts to which the speaker has had access, the vegetable matter in the water does not diminish in like ratio. The distant or middle portion of such a body of water, when compared with intermediate points near the shore, will show a quite regular change or gradation in the amount of vegetable matter. Further than this, as shown by Rafter's examinations of Lake Erie water, the quantity of vegetable matter in the middle of the lake increases quite rapidly with the depth, and this is a natural inference if we attribute the reduction in amount of vegetable matter to the action of the atmosphere upon the water, although at certain seasons, when temperature controls the vertical movement of the water, this may not hold true.

It would seem, then, that a natural process, partially bacterial and increased or perfected by the continued movement of water through the soil, effects the decolorization of water on land, and that a natural process, something like the ozone process, is to quite a degree effective without great assistance from bacteria when the water is exposed to the atmosphere in a considerable body, as in a lake or large reservoir.

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RAILROAD DISCRIMINATION AGAINST NEW YORK, AND THE REMEDY.*

By Abel E. Blackmar, Esq.

Presented May 1st, 1901.

WITH DISCUSSION.

Railroad discrimination may be divided into two classes:

The first is that which consists of special rates, drawbacks and secret agreements, giving to certain shippers advantage over others. This is illegal. It is injurious to trade at large; it aids the strong against the weak, creates conditions hostile to proper development of commerce, and repudiates the duty of uniform service which the railroads owe to the people; but it is not necessarily or usually discrimination against a locality. It accompanies free competition among the roads, and in the end leaves each locality with the benefit of its natural and acquired advantages.

The second class of discrimination consists of differences in published rates, made for the purpose of influencing the direction of traffic, and so favoring or injuring localities. This is not necessarily illegal

* The statements contained in this paper apply to differentials up to the early part of the year 1900.

As was said by the Interstate Commerce Commission, in Produce Exchange *vs.* The Baltimore and Ohio Railroad Company:

"Railway companies are not prohibited by the third section from preferring one locality to another, unless the preference amounts to an undue or unreasonable one. * * * Railway competition may, therefore, excuse the giving of a preference to a particular locality or a particular commodity, provided the interests of the public are not unduly sacrificed to those of the carrier."

We must accept the proposition, therefore, that a discrimination against a locality in the interests of a carrier is not necessarily unlawful. It is of such discrimination that this paper treats.

On April 5th, 1877, an agreement was signed by William H. Vanderbilt, H. J. Jewett, Thomas A. Scott and John W. Garrett, representing the New York Central, the Erie, the Pennsylvania, and the Baltimore and Ohio railroad companies. This agreement established a system of differential freight rates governing all traffic, both east-bound and west-bound, moving by rail between the cities on the Atlantic seaboard and a territory bounded on the south by the Ohio River, on the west by the Mississippi, and on the east by an imaginary line drawn through Toronto, Buffalo, Pittsburg and Parkersburg. All traffic destined to or originating in territory west of the Mississippi was also subject to this agreement, and it has since been extended to other trunk lines and to seaports like Newport News and Norfolk, which have since become important. This system of differentials has continued, with some modifications, substantially unchanged until to day. It governs the rates on all classes of freight and commodities, and regulates the movement of all grain and grain products of the United States, except those moving to the Gulf ports or the Pacific Coast.

In determining the freight rates, the New York-Chicago rate is taken as the basis, because those points are the most important shipping centers, and the rate between them is, during the period of open navigation, in direct competition with the lake and canal route. The rates between New York and all other points in the territory are agreed percentages of the New York-Chicago rate. For instance, the rate from New York to Detroit is 78% of the New York-Chicago rate; to Indianapolis, 93%; to Cincinnati, 87%; to Erie, 60%; to St. Louis, 116%, and so on. To determine the rate between such points and the other seaports on east-bound freight, 2 cents per hundredweight is

deducted from the New York rate to find that to Philadelphia, and 3 cents to find the rate to Baltimore, Norfolk and Newport News. On east-bound traffic for export, Boston takes the New York rate. On west-bound traffic the differential is the same as upon the east-bound on lower classes of freight, and greater on higher classes. In this manner, rates from every point in that territory to every point on the seaboard are constructed by the use of these percentages and differentials, and vary as the basic rate from New York to Chicago varies.

As was said by Commissioner Fink in his report dated December 1st, 1881:

"The agreement of April 5th, 1877, was made with the view of equalizing the aggregate cost of rail and ocean transportation between the competing points in the West and all foreign and domestic points reached through Baltimore, Philadelphia and New York."

This is a confession that the object of the agreement was to give to the other ports and the railroads leading thereto such advantages in freight rates as would neutralize all the natural and acquired advantages of New York, and put all the ports on a competitive equality as to traffic for export. This confessed purpose of the differential fell short of the truth, for they have always been much greater than the ruling difference in ocean freights, and have given to other ports advantages which have enabled them to compete successfully with New York and divert a considerable amount of the traffic properly tributary thereto.

To more fully carry this purpose into effect, the original agreement of April 1st, 1877, provided:

"That the cost, to the shipper, of delivering grain at each port from the terminus of each of the roads to the vessel in which it is exported, as well as the number of days' free storage thereon allowed, shall be the same."

As the whole differential system is based on a variable unit always in competition with the lake and canal, so the regulations regarding elevator, lighterage and transfer charges were devised to meet water competition. The great ocean steamers in New York receive their berth freight at their docks. In delivering to such docks, the canal boat, being a moving storehouse, has the advantage of the railroad with a fixed terminus. The railroads, therefore, provided for free lighterage, to put themselves on a competitive equality with the canal. They agreed with the Floating Elevator Company that the same transfer

charge should be imposed on canal grains as, by agreement between themselves, they imposed on grain carried by rail, and they placed an arbitrary prohibitive charge of 1 cent a bushel on grain loaded for export at their own elevators, to maintain the condition of equalized charges in order that the differential might have full effect.

Having arranged for this self-adjusting system of rates which created a practical consolidation of all the trunk lines participating in the grain trade moving to the Atlantic seaboard, and having established a system of excessive transfer and terminal charges, which, as to rail-carried grain, was a practical addition to freight charges, the railroads began the contest for the carrying trade with the Lakes and canal. The problem before them was a difficult one: It was to preserve to the railroads having their termini in New York their fair percentage of the traffic, and at the same time to divert to the roads tributary to Boston, Philadelphia, Baltimore, and afterward Norfolk and Newport News, such an amount of freight as would satisfy their claims. This purpose was the advancement of railroad interests alone, without regard to the rights or interests of the competitive localities. As New York controlled the trade, this scheme could only be carried out at the expense of the trade of New York, and by the limitation or destruction of the traffic by lake and canal.

In the contest with the Lakes, the railroads, although unable to compete in the rate per ton per mile, had some powerful weapons. All grain moving down the Great Lakes is brought to the Lake ports from the interior by rail. When grain is once loaded into cars, its tendency is to remain there until it reaches its ultimate destination. By high local rates to the Lake ports and low through rates to the seaboard, the railroads attempted to prevent the transfer of the grain from their care to the Lake carriers.

According to the testimony of Mr. Hayes in 1874, before a select committee on transportation routes to the seaboard, the through rail rate to New York from points of origin in the corn and wheat belt, was 1¼ cents per ton-mile, while the rate to Chicago from the same point was from 2.1 to 2.3 cents per ton-mile.

In 1872 the railroads had practical control of all the territory in Illinois and Indiana within 60 miles of Lake Michigan, and, by discrimination against the Lake ports, were pressing forward their sphere of influence. For some time the railroads seemed to be succeeding in

their contest with the Lakes, but since the early eighties the natural superiority of the waterways has asserted itself.

In 1881 the railroads had secured 94% of the flour traffic from Chicago, as against 4% carried on the Lakes.

In 1882 the shares of the railroads fell to 79%
" 1886 " " " " " " " 62%
" 1889 " " " " " " " 48%

From that time the percentage has not substantially fallen off; while in one year—1898—it materially increased.

In 1874 the railroads carried 37% of the wheat from Chicago; and the Lakes, 63 per cent.

In 1878 the railroads secured 44%
" 1879 " " " 41%
" 1897 their percentage fell to 32%

In 1885 the railroads carried 80% of the oats from Chicago, as compared with 20% by Lake.

The percentage of the railroads has fallen until 1897, when they secured only 52 per cent.

These are statistics of the Chicago trade where the conditions favor the railroads; the actual increase of Lake transportation of wheat and flour has been greater, as the flouring industries in northern towns have developed, and the wheat belt tributary to the Lakes has shifted northward toward Minnesota and the Dakotas.

As the corn belt is further south, the railroads have been more successful in maintaining their share of the through corn traffic.

Statistics demonstrate the growing supremacy of the Lake traffic: In 1885 the record of entrances and clearances at the United States ports on the Lakes showed a traffic of upwards of 19 000 000 tons; by 1890 this had reached 37 500 000 tons, and by 1896 the total traffic was 52 000 000 tons. According to the report of the Roosevelt Commission on canals, 60 000 000 tons will not over-estimate the freight traffic on the Lakes in 1898, or a ton-mileage of 42 000 000 000 for the year 1898, exceeding the total ton-mileage of railroads in the district from the Missouri River east to Buffalo and Pittsburg. The greater part of this traffic is in the coarser products, and the natural destination of the east-bound traffic is the Port of Buffalo, and the natural outlet the Erie Canal.

The success of the Lakes in this struggle shifted the point of battle between the railroads and the waterways from Chicago to Buffalo. As was said in the "Monograph on the Grain Trade," issued by the Treasury Department, in January, 1900:

"While the Port of Buffalo is receiving ever-increasing quantities of grain, and while canal freights have been reduced, tolls abolished, and elevating and storage charges not only lessened, but in some cases temporarily abolished (the elevating being done gratis), the canal is becoming less important, while the Lakes are maintaining their own, and even regaining their old supremacy. The break in the chain of water communication, however, is not in the West, but at the point where lake and canal meet at Buffalo."

To meet the changing conditions caused by the reviving commerce on the Lakes and prevent the commerce seeking an outlet down the canal, the railroad companies, in 1895 and 1896, imposed a differential, first of ½ cent and then 1 cent a bushel upon ex-lake grain, in favor of Philadelphia and Baltimore, and against New York. As the Buffalo-New York rate was made in competition with the canal, that to Philadelphia and Baltimore was amply effective to produce the diversion. This is a discrimination, not only against the Port of New York in favor of Philadelphia and Baltimore, but against Buffalo in favor of Erie and Fairport.

The whole differential system, including the all-rail differential, the lake and rail differential, and the maintenance of excessive transfer, elevator and storage charges, had, as has been seen, two objects:

First.—Solidifying all railroads in their competition against the waterways, natural and artificial.

Second.—Diverting to other places trade properly tributary to New York, and so distributing the traffic as to satisfy competing roads.

In their first purpose they have largely failed in their struggle with the Lakes; but they are succeeding better against the canal.

What has been the effect of these discriminating differentials upon the trade of New York?

In the early seventies, New York controlled between 70 and 80% of the export trade passing through the six ports of Boston, New York, Philadelphia, Baltimore, Norfolk and Newport News, and an even larger percentage of the import trade. It had many advantages which had enabled it to secure and retain this trade. The Erie Canal was then bringing to the Port of New York as much grain

and flour as all the railroads combined, and it was bringing about one-third of all the grain which was received at all the six Atlantic seaports. Foreign trade had been firmly established by many and well-recognized lines of ocean steamers which resulted in favorable ocean freight rates. Conservatism and the force of habit in trade favored that port, storage facilities permitted almost unlimited receipts, and, lastly, the freight rates themselves were so much higher than at present that the relative burden of the differentials was much less. For the four years preceding the differential agreement, namely, 1873, 1874, 1875 and 1876, there was received at New York 57.2% of all the wheat, flour and grain received at the six Atlantic seaports. During the last four years the receipts have fallen to 36.9 per cent. During the four years preceding the differential agreement, New York had 67.4% of all the exports of such products passing through these six ports. During the last four years New York's percentage has fallen to 34 per cent. This is a decrease in the percentage of receipts at New York of 35.6%, and a decrease in the exports of nearly 50 per cent. This decrease in the percentage of the traffic passing through the Port of New York has been continuous and progressive. Year by year the ports, especially those to the south, which have the larger differential, have encroached upon the trade of New York. For many years the conservative tendencies held a large portion of the trade of New York, but with the elimination of the canals, with the increasing facilities for handling the traffic at other ports, with the construction of new roads which are naturally tributary to other ports, with the cheapening of the price of breadstuffs and the decrease in freight rates, the diversion of traffic has grown from year to year more serious. It was felt first in coarser materials, especially in grain and grain products, but is beginning to be followed by the diversion of trade in other commodities and by decrease in imports.

This is not a question affecting the Port of New York alone; this traffic is diverted from the State. As has been pointed out, the system of differentials which affects grain moving from the Lake ports by rail to the Atlantic Coast is aimed directly at the trade of Buffalo in favor of Erie for the Philadelphia trade, and Fairport for the Baltimore trade; neither are interior points like Rochester and Syracuse unaffected. The rail rates on coarser freights from Syracuse and Rochester to the seaboard are the same as from Buffalo. This subjects such

freight to the influence of the ex-lake differential, and, practically, as far as freight rates upon this class of products are concerned, brings Duluth within 1 cent a bushel as near Philadelphia and Baltimore as Syracuse is to New York.

Is there any remedy, and, if so, what?

For twenty-three years the merchants of New York have been attempting to obtain relief by application to the railroad companies, but without success. During all this time the question has been agitated in every manner possible; every effort has been made to obtain relief, and the result has been failure. After the establishment of the Interstate Commerce Commission, it was hoped that relief might be had on the theory that the differentials were an undue and unreasonable prejudice and disadvantage to the "locality" of New York in favor of other competing localities. Proceedings seeking relief were instituted by the New York Produce Exchange, but they resulted only in obtaining a judgment that the differentials were not illegal. After this failure, strenuous efforts were made in the direction of decreasing terminal charges, storage charges and inspection fees, but these resulted in nothing permanent.

The remedy is not with the railroads. There were times when the New York Central and Erie, which were distinctively New York roads, were hostile to the differentials. On June 3d, 1880, the New York Central road withdrew from the differential agreement, claiming that it was unjust to the Port of New York. A war of rates immediately followed, and lasted until the Report of the Advisory Commission, on July 20th, 1882, recommending the continuance of the differential. The rate war whipped the New York Central back into line, and forced its acquiescence in the differential system. Since that time an era of consolidation of railroad interests has arisen, and it is impossible to say, with any assurance, that the success of any railroad system depends on the prosperity of any single port. The motive which once was powerful enough to induce the New York roads to brave a rate war in the interests of the commerce of New York is gone, or has become very much weakened. If, before the time of railroad consolidation, the New York roads, then interested solely in New York commerce, relatively stronger than at present, with less competition from the southerly roads, were unable to effect the abrogation of the differential, what possibility is there that such result can be now reached?

There are also substantial reasons for the apathy of the New York roads in their efforts to protect New York commerce. Since 1873 the excess in receipts of wheat, corn and flour over exports at the Port of New York was 707 218 133 bushels; this seems to represent the domestic consumption. The differential at 2 cents a hundredweight upon this amounts to about $8 000 000. The differential at 3 cents a hundredweight is $12 000 000, or upwards of $444 000 a year. If all this produce were brought to New York by rail, under freight rates governed by the now existing differential, the city would pay in increased freight over the Baltimore rate upon the wheat, flour and corn there consumed each year, an amount which about equals the annual expenditure for the maintenance and operation of the Erie Canal, which has averaged, for the ten years preceding 1899, about $445 000 per year. The total receipts of wheat, corn and flour at New York since 1872 have been 1 589 448 207 bushels. The 2-cent differential on this has amounted to about $18 000 000; the amount of the 3-cent differential is about $27 000 000; or, an average of upwards of $1 000 000 per year. That portion of this freight which has been carried upon the Erie Canal was not, however, subject to this tax. Although, in the year 1899, less than one-tenth of the receipts of wheat, flour and corn at the Port of New York came by canal, yet, during the whole period from 1872, about one-third of the receipts were by canal. This portion of the traffic was not subject to the differential, and the total burden upon the Port of New York, through the operation of the differential on east-bound wheat, corn and flour alone, since 1872, has been, therefore, about $18 000 000, or upwards of $666 000 yearly. This is a tax which has been paid to the roads carrying to New York City, for the purpose of inducing them to consent to the diversion of the trade to the more southerly outports by the operation of the differential. It is an ingenuous method which enables the New York roads to exact from New York commerce the price of their acquiescence in its destruction. The figures here given represent the amount of the differential on wheat, corn and flour alone. The total amount of the differential upon all traffic, both east-bound and west-bound, through the Port of New York is immensely greater.*

It is also a significant fact that this decrease has not been at the expense of the railroads. The amount of wheat, corn and flour re-

* On 3 816 064 tons of flour, grain and provisions reaching New York in 1896 by rail, the 3-cent differential amounted to $2 289 638.

ceived at New York by rail during the four years preceding the differential agreement, viz., 1873, 1874, 1875 and 1876, was 30.2% of the total receipts at the six Atlantic seaports, and during the last four years 1896, 1897, 1898 and 1899, it had increased to 32.5 per cent. The percentage of grain carried by rail to the Port of New York has been almost a constant factor for the last twenty-seven years. The railroads, notwithstanding the differential, have maintained, not only the absolute amount of traffic carried, but have secured their percentage of the increase. All the loss of this traffic to the Port of New York is represented in the loss of the canal-carrying trade. The differentials have been potential in giving to the other ports that portion of the grain trade which used to be carried by the Erie Canal. In 1893 the receipts at New York were 61.4% of the total amount of the receipts at the six Atlantic seaports; of this the canal carried over one-half, or 32.6%, and the railroads 28.8%. In 1899 the receipts at New York had fallen to 36.2%, of which the canal carried only 3%, the railroads actually increasing their percentage to 33.2.

It is found, then, that the railroad companies have no motive for protecting, or power to protect, the commerce of New York against the inroads made by the differential system in favor of other ports; that existing conditions do not permit an abrogation of this discrimination by the railroads themselves, and that practical experience for twenty-seven years proves that it is impossible to anticipate any help from legislative bodies or judicial tribunals. The differentials may at some time be voluntarily abolished by the railroad companies. They have been during the last year decreased upon grain intended for export; but, if abolished by the railroads, it will be when they have served their full purpose, when the canal has been entirely eliminated as a factor, when the other ports have established direct relations with foreign ports by their lines of steamships, and by this means secured favorable ocean berth rates, and when their harbor accommodations and facilities for handling products have been improved so that they can hold their own against the Port of New York, when what the Baltimore and Ohio, the Pennsylvania, the Chesapeake and Ohio, and the Norfolk and Western are pleased to consider their fair proportion of the traffic has been permanently secured to them and can be maintained without the aid of the differentials, then the railroad companies may voluntarily abolish them, but it will not be before

that time. If any relief is expected, it must be sought in another direction. Nor will palliatives like diminution of port charges, cheaper docking facilities, reduction of elevator charges, or abolition of lighterage, permanently provide relief. These reforms are useful and necessary, but not sufficient, for they will be met with a proportionate decrease in other ports, leaving the influence of the differentials intact.

But there does exist within the power of the people of New York a remedy which is sure and effective. There is a transportation agency which never can be interested in any termini except New York or Buffalo, nor in any interior cities, except such as Rochester, Syracuse, Utica and Albany. This agency can never make agreements with railroads to divert traffic to any other ports. It can be a party to no scheme of imposing and maintaining arbitrary charges for lighterage or elevators. It can serve only the people of New York, and it can restore to New York in a decade the trade which has been laboriously and steadily diverted by the railroad companies. This agency is an improved and enlarged navigable waterway from Lake Erie to the Hudson River. If such a waterway were built, and if it secured a reasonable proportion of the traffic which is properly and naturally tributary to New York, self-interest in the railroads leading to New York could be relied upon to continue to secure their own percentage as against roads leading to other points, in addition to that carried by the canal.

In proceedings taken by the New York Produce Exchange, against the railroad companies, before the Interstate Commerce Commission, seeking the abrogation of the differentials, Mr. George R. Blanchard, Commissioner of the Joint Traffic Association, was placed upon the stand, and testified that in the year 1896 the business by canal was approximately 6% of the total receipts of wheat, corn and flour at the six Atlantic seaports, and 17% of the total receipts at New York; "that one-half of that proportion or less might, and probably would, determine the rail rates from Buffalo to New York and incidentally from all lake ports to all seaport cities." Such, in the estimation of railroad men, is the present power of the antiquated, neglected and obsolete waterway, a power, the exercise of which is of infinite value to the people, and is even now the flaming sword in the path of railroad monopoly. In view of this it is childish to figure upon the cost of moving the 3 000 000 or 4 000 000 tons of freight actually carried by the Erie Canal. It is a

limited vision which can see no other use of the canal. As important as this may be, the indirect influence of the canal is much greater. Antiquated and ineffective as are the present canal methods, representing conditions of nearly eighty years ago, with boats carrying 320 tons, with the waterway insufficient even for them, with its mule power, pike poles and snubbing posts, full of eel grass, crawling along the hill-sides, the Erie Canal is to-day a regulator of freight rates, and sets a limit upon railroad exaction, not only for the State of New York, but for all the grain which is grown in the United States, whether it finds its outlet at the Atlantic seaports, at the Gulf ports, or possibly at the Pacific seaports.

In the report of the Canal Auditors of 1883, there is the following:

"It is nearly sixty years since the Erie Canal was completed and opened for navigation between Lake Erie and the Hudson River, and all the primitive devices for operating it are still employed. When we reflect that the whole system of operating the locks and moving the boats has undergone no change, it is not surprising that the canals have become the subject of ridicule. The fact that the canals continue to be employed under such conditions is a strong proof that to a certain extent they are a necessity."

That was written seventeen years ago, and the present writer knows of no substantial improvement since.

If the present "ditch," so-called by its enemies, if this "hindrance to commercial prosperity," to use the heading of a paper read before the State Commerce Convention at Utica in 1899, has such a power to-day, what limit could be put upon the power of a canal such as is recommended by the Canal Commission of Governor Roosevelt, to vitalize and develop commerce and, incidentally, manufacturing industries?

Such a canal would be an effective remedy for railroad discriminations.

First.—It would reduce freight rates so low that a differential would be impossible. The most that the most sanguine and irresponsible advocate of railroad as against canal transportation has claimed is that it is possible for railroads to compete with the canal According to the United States Bureau of Statistics, the rate per ton-mile on the New York Central is 6.76 mills; on the Erie, 6.09 mills; on the Baltimore and Ohio, 5.29 mills; on the Pennsylvania, 5.61 mills; on the Lake Shore, 5.38 mills; on the Chesapeake, 4.19 mills; and on the Norfolk and Western, 4.46 mills. The writer has examined the statistics of

upwards of a hundred roads and finds none lower. Grain has been carried at lower rates, and, especially from the Lakes to the seaboard during canal seasons, rates have been made as low as 2.50 mills per ton-mile; but these are rates created by canal competition, and it has never been claimed that they are profitable to the railroads, except to aid in the destruction of the canal. These low rates granted from Buffalo to New York are Grecian gifts like free elevating of canal grain by railroad elevators done to drive canal elevators out of business. But even for the sake of argument concede the possibility of railroad competition with a rate of 0.52 mill per ton-mile, it is obvious that no differential could be placed under this. A sufficient reduction of canal freights means the end of the differential discrimination against New York commerce; with such a canal a freight war would not follow an abrogation of the differential, especially in coarser freight; for the futility of such a rate war would be obvious to all, and even railroad companies do not commit suicide. The roads carrying to New York in competition with such a canal would be obliged to demand and not accord differentials, and the commerce of the great Northwest would once more flow to and fro through its natural channel, the gap through the Appalachian Chain, along the course of the Erie Canal and through the Hudson River, and every city, every village and every county along its course would be enriched by the stream of commerce as it flowed.

Second.—It would put an end to exorbitant transfer charges which have been established by the railroad elevator combination. If the grain which arrives in Buffalo, together with that which would be attracted there by the abolition of the differential, should find an outlet down the new enlarged canal, it would burst through any attempted system of extortion by transfer charges, elevating, storage or inspection, as surely as a mountain torrent will burst through a dam of bulrushes.

The railroads cannot afford us a remedy. They are powerless before their competitors, and have to agree with them, by the way. The reported talk of a railroad president about carrying grain from Buffalo to New York free, were he given the price of canal enlargement, is idle. If it were possible, and he were willing to do so, it would not be permitted by competing roads. Legislation is futile before the development of trade under natural laws. The only remedy is to create such conditions that the natural operation of the laws of trade prohibit the discrimination.

DISCUSSION.

H. S. HAINES, Vice-President, Am. Soc. C. E.—This paper does not treat the subject of railroad differentials as an engineering question. It is an examination into railroad rates on grain for export, as affecting the commercial interests of New York City against other North Atlantic ports, and is only capable of discussion from this point of view. Mr. Haines.

The author enters upon his argument with a reference to the New York-Chicago rate as the basis for a sliding scale to which may be referred rates by the trunk lines between all points in certain territory and North Atlantic ports. Rates from the same territory to South Atlantic and Gulf ports are also virtually referred to the same basis, and rates from contiguous territory are indirectly governed by it. Therefore, upon the rates from Chicago to New York, upon Western products, are based the rates upon the same commodities from the entire Mississippi Valley to tide water. Every fluctuation in these rates affects that enormous volume of traffic, and great interests, involving producers, merchants and intervening means of distribution and transportation, are at the mercy of secret differentials in the rail rates from Chicago to New York. So that, in discussing railroad differentials, this aspect of the question should not be obscured.

So far as this sliding scale affects export traffic, it becomes an element in making joint rail and ocean rates through each port partaking of the traffic. Therefore, with the rail rates established on any defined basis, the fluctuation in such joint rates is restricted to the ocean rates. Here, free play is given to the efficient operation of the advantages which each competing port may have to offer. The purpose of the sliding scale is briefly and correctly set forth in Commissioner Fink's report of December 1st, 1881, viz., to equalize the aggregate cost of rail and ocean transportation. Taking a broad view of the subject, covering the welfare of all the interests involved, railroads, ports and steamships, this purpose is to conduct the export traffic under conditions as to which all the parties interested shall be equally well informed. It is not just, therefore, to the parties to the agreement to say that its purpose, as set forth by Commissioner Fink, fell short of the truth.

The author's strictures upon it are two: That the agreed differentials have always been much greater than the ruling differences in ocean freights; and that they have given other ports advantages which have enabled them to compete successfully with New York and to divert a considerable amount of traffic properly tributary thereto.

His first point, that the agreed differentials have always been much

Mr. Haines. greater than the ruling difference in ocean freights, is a question of fact. That fixed rail differentials should sometimes vary from the difference in ocean rates respectively prevailing at the ports is unavoidable, but it is not to be presumed that the lines terminating in New York would have consented that they should always have been greatly adverse to the port in which they were interested. The burden is consequently upon the author to prove that such an inequitable difference has always existed.

His next assertion, that the differentials have given to other ports advantages which have enabled them to compete successfully with New York, does not cloud the purpose of the differentials if by successful competition is meant an equitable competition. The purpose only becomes inequitable to the Port of New York when it tends to divert from that port an undue amount of the traffic subject to competition. This the author assumes to have been the concealed purpose of the differential agreement, and he proceeds to establish his thesis by stating the disadvantages which export traffic through the Port of New York has thereby suffered. He criticizes the provision that the cost of transfer from car to ship, including free storage, shall be equalized as between the terminal ports. As a general proposition this accords with the purpose of the agreement, which was, fundamentally, to limit the variations in the joint rail and ocean rate to that element which was controlled by ocean carriers, free to load at any port where business offered. But the writer asserts that this provision was devised to meet water competition. If the provision were equitable, as applied to ports at which no inland water competition existed, why should it become inequitable because it included a port at which such competition did exist?

Here he narrows the discussion to the competition of the rail lines terminating in New York with the lake and canal service, a local competition which could not militate against New York interests. He says that the canal-boats had the best of the railroads in making deliveries, and that the latter endeavored to put themselves upon an equality with the former by providing free lighter delivery. How was this injurious to the Port of New York? But they also secured an equal elevator charge for grain, whether by rail or canal. To this also the author objects, although he refers to both as a system of excessive transfer and terminal charges which was a practical addition to the rate on rail-carried grain. He now assumes that what he properly calls a self-adjusting system of rates created a practical consolidation of all the trunk lines participating in the grain trade moving to the Atlantic seaboard. If he meant to imply, by the expression "practical consolidation," an indifference on the part of the railroad managements as to the route by which export grain should reach the seaboard, his subsequent argument demolishes the assumption. For

he proceeds to unfold another opinion as to the purpose of the differential agreement, viz., that it was intended to preserve to the railroads having their termini in New York, their fair percentage of the traffic, and at the same time to divert to the roads tributary to Boston, Philadelphia, Baltimore, and afterward to Norfolk and Newport News, such an amount of freight as would satisfy their claims. Mr. Haines.

The twist in this statement is in the use of the word "divert." The agreement was really intended to prevent the diversion from New York to the other ports, by secret differentials, of a very large part of the export grain trade which New York had hitherto enjoyed, but which that port no longer controlled. The world had moved since the construction of the Erie Canal. New York was no longer the only port by which the products of the great West could reach European buyers. The grain-growing area had spread far away from the Great Lakes, and much of it was nearer tide water at the Delaware and Chesapeake Bays, and even in the Gulf of Mexico, than in New York Harbor. It was this changed condition of affairs which forced the rail lines terminating in New York to accede to the differential agreement, and every step that they took to limit their own freedom of action was taken to preserve and protect the commerce of New York City. The lake and canal rates formed undoubtedly the basis on which the differentials were adjusted. If those rates were lower than the recognized cost of the same service by rail, how could the rail lines compete for the traffic, except by offering advantages which the lake and canal route did not offer in connection with a lower rate? As to elevator and transfer facilities in New York Harbor, they were as available for shipments by canal as by rail. There was nothing to prevent canal-boat owners from owning elevators or lighters.

How, then, was the lake and canal traffic limited or destroyed? The author now says, by high local rates to Lake ports and low through rates to the seaboard, that, in 1874, from points of origin in the corn and wheat belt, the rail rate to New York was 1¼ cents, while to Chicago, from the same point, the rate was from 2.1 to 2.3 cents per ton-mile. Comparisons based on statistics twenty-seven years old, however valuable to the historian, have no direct bearing on this discussion. Yet, what has been the effect of the differential agreement, even where the conditions favored the rail lines, that is, from Chicago?

The writer gives the proportionate tonnage from Chicago by the rail lines as follows:

Flour traffic, 1881, 94%; 1882, 79%; 1886, 62%; 1889, 48 per cent. Since that year no material change.

Wheat traffic, 1874, 37%; 1878, 44%; 1879, 41%; 1897, 32 per cent.

Oats traffic, 1885, 80%; 1897, 52 per cent.

From these statistics it appears that from Chicago, the point most favorable to the rail lines, and, notwithstanding the differential agree-

Mr. Haines. ment, their proportion of the flour traffic has decreased nearly one-half; they have been unable to retain the proportion of the grain trade that they carried in 1874, and they now carry one-half instead of four-fifths of the oats, and, as the flouring industries in northern towns have developed and the wheat product of Minnesota and the Dakotas has increased, so has the Lake transportation of wheat and flour increased. The Lake traffic has increased from 19 000 000 tons in 1885 to 52 000 000 tons in 1896. The differential agreement, then, has not had the effect of diminishing the traffic by the Great Lakes.

But the author says that the point of competition has been shifted from Chicago to Buffalo; that there has been an enormous increase in traffic destined for points east of Buffalo, and, of course, largely for New York, and that the competition between rail and canal at Buffalo is mainly for New York traffic. This traffic was not subject to the differential agreement which he says was intended to divert traffic from western territory properly tributary to New York. He now refers to another agreement made in 1895 or 1896 relating to grain traffic from Lake ports to the seaboard which established a differential of 1 cent per bushel in favor of shipments from Erie or Fairport to Philadelphia or Baltimore, as against the traffic from Buffalo to New York by rail. Discarding the discussion of the differentials covering traffic from western territory he reduces the issue to the differentials upon grain from Buffalo, Erie and Fairport. These he considers as open to the same objection: That they are intended to strengthen the rail lines in their competition with the canal to divert trade properly tributary to New York, and to distribute traffic so as to satisfy the competing roads. It is neither immoral nor unfair for railroads to compete with canals, and, so long as there is no combination to advance rates, neither producer nor consumer is unfavorably affected by such competition.

The issue is accordingly narrowed down to the diversion, to Philadelphia and Baltimore, of Lake traffic properly tributary to New York. What Lake traffic should properly pay tribute to New York? What other traffic than that which cannot avoid doing so; that is, traffic in commodities intended for consumption or distribution at New York City? Any other traffic is purely competitive, and so long as that competition is fairly conducted, New York interests have no cause for complaint against rail lines terminating at other ports. The responsibility for maintaining New York in the front rank depends upon the rail lines terminating there. Against them should the complaint be directed, accompanied by the evidence that they have failed to do their duty in this respect.

The author asserts that in the early seventies New York controlled between 70 and 80% of the export trade passing through the North Atlantic ports. Compare the track mileage in the early seventies con-

trolled, respectively, in the interest of each of these ports, with the mileage as controlled to-day, and it will be seen that the situation has greatly changed as to competition in the grain-producing territory. That territory itself has been continuously extended in a direction favorable to short hauls to the Delaware and Chesapeake Bays, and also to the South Atlantic and the Gulf ports. Under such conditions it is not to be expected that New York should continue to control nearly 80% of the total export trade, nor that the Erie Canal should continue to control one-third of all the grain received at these ports. The rates by rail have been largely reduced since the early seventies. The all-rail grain rate from Chicago to New York, of 1½ cents per ton-mile, quoted by the author, has been reduced to less than ½ cent, and the canal-boats have not been able to meet this reduction. In his comparisons the author deals in percentages, not in quantities. To sustain his thesis that the rail lines terminating in New York have failed to keep that port in the front rank in the grain traffic, he should have compared the movement of the export trade in tons, distributed as to areas of origin, and should have entered more into details in support of his assertion, that, of the largely increased Lake traffic, an undue proportion has been diverted from Buffalo to Erie and Fairport. Mr. Haines.

Though he has not presented such statistics, he assumes that he has established his charge of discrimination, and calls for a remedy. He says that for twenty-three years the merchants of New York have called in vain for relief upon the rail lines terminating there. If the managements of these lines have not responded satisfactorily to such appeals, they have failed to meet their responsibility through intention, neglect or inability, and, when the charge is clearly established, the supporting evidence should show to which of these causes such failure is due. But if the merchants of New York are convinced that, for any reason, the rail lines terminating at that port have not kept it in the front rank, they are right to seek a remedy. When these lines were relatively stronger than at present, and interested solely in New York commerce, they attempted to abrogate the differential agreement, but were whipped into line after a war of rates that lasted for over two years. The Interstate Commerce Commission has decided that this agreement is not illegal, and strenuous efforts to decrease the terminal charges have been unsuccessful.

With these conclusions, the author connects an estimate of the burden which the traffic of New York has had to bear, by reason of the differentials upon grain and grain products for domestic consumption and distribution; a burden which he estimates variously at 2 cents and at 3 cents a bushel, and averages at about $666 000 per annum. He calls this a tax, paid to the roads terminating in New York City, to induce them to consent to the diversion of trade; an exaction, by the New York roads upon New York commerce, of the price of their

Mr. Haines. acquiescence in its destruction. He has just asserted that the Interstate Commerce Commission decided that the differentials were not illegal, and that, after fighting them for two years, the New York roads had been eventually whipped into line. Why, then, charge these roads with adopting the differentials as one ingenious method for exacting from New York commerce the price of their acquiescence in its destruction?

But he goes on to say that, notwithstanding the differentials, the New York roads have not only maintained the absolute amount of tonnage to New York, but that they have also secured their fair percentage of the increase. Why say "notwithstanding the differentials"? Why not by reason of them? If, as he says, they have secured their fair percentage of the increase, why hold them accountable for the decreased traffic by canal? At this point he absolves them. He admits that the differentials have recently been reduced and that they will probably be abolished, as other ports secure favorable ocean rates and improve their harbor facilities. This in itself would seem to be a remedy for the alleged railroad discrimination, but the author relies solely on improving the Erie Canal. He thinks that if the canal were enlarged and the Hudson River navigation improved, and if the water route thereby secured a reasonable proportion of the traffic properly and naturally tributary to New York, the railroads could be relied on to secure their own percentage of the competitive traffic. Into the engineering questions involved in this proposition he does not enter, nor are they pertinentto his thesis of "Railroad Discrimination against New York and the Remedy."

An examination of this paper elicits the proposition that, while the Lake traffic has increased enormously, some portion of it, properly tributary to New York, has been diverted, *via* Fairport and Erie, to Baltimore and Philadelphia, by means of lawful differentials, but that the portion so diverted has not been taken from the rail lines terminating in New York. The Erie Canal has failed to carry such a percentage of the traffic as would make it an efficient remedy against railroad discrimination. The improvement in this waterway must be of a character that would bring about the following results:

First, it must reduce rates to a point below which competing rail lines could not establish rates. The author fixes the lowest known rate by rail between the Lakes and the seaboard at 2½ mills per ton-mile. His idea is that the water route should furnish transportation at a rate of about 0.052 mill per ton-mile, to prevent the possibility of rail differentials, and that then the commerce of the Great Northwest would once more flow through its natural channel, the gap through the Appalachian chain.

Second, that the construction of this enlarged waterway should

put an end to exorbitant transfer charges upon grain arriving at Buffalo, furnishing such an outlet for it as would enable it to burst through any system of extortion. Mr. Haines.

To these conclusions, so far as they bear upon the prosperity of the City of New York, no person whose interests are in line with them would be disposed to take exception, nor has the writer undertaken to discuss them from any other point of view than that the grain differentials have diverted from New York a considerable amount of traffic properly tributary to that port, and that it was in the power of the rail lines terminating in New York to abolish these differentials of their own motion. If freight can be carried from Buffalo to New York at a rate of 0.052 mill per ton-mile, it will certainly divert to the latter port much of the Lake traffic now taking other routes. Whether, as the author suggests, it was idle talk on the part of the railroad president who proposed to carry grain free from Buffalo to New York for the price of the canal enlargement, that is another story.

WALTER G. BERG, M. Am. Soc. C. E.—The title of this paper does not indicate the general and broad question evidently involved, namely: "The Reasons for the Decline of the Commerce of the Port of New York, and the Remedy." The paper is in reality a one-sided argument, offering only one reason for the decline of the commerce of the Port of New York, consisting of a summary of the past history of the question of rail differentials and a general arraignment of the railroads as the sole cause of the decline. Mr. Berg.

This portion of the paper is practically an abstract of the arguments, testimony and briefs presented in 1897 in the suit of the New York Produce Exchange *vs.* Baltimore and Ohio Railroad Company and thirty-one other railroad companies before the Interstate Commerce Commission, the complainants being represented by the law firm of Baldwin & Blackmar. This case was decided April 30th, 1898, by the Interstate Commerce Commission, the decision being: "The complaint is dismissed without prejudice." The Commission went into the questions involved, *i. e.*, the legality and the reasonableness of the existing rail differentials, with great detail, holding meetings in various cities and taking testimony of all kinds, and finally ren dered a most exhaustive review and decision covering all the phases of the question. While the author quotes certain clauses of the decision, the following additional clauses will serve to illustrate the scope of the inquiry and the importance of this adjudication of the question of New York rail differentials by the proper court of inquiry established by Congress:

"Railway companies may make whatever rates, form whatever lines, and establish whatever differentials they deem best for the purpose of securing and conducting transportation, provided the just interests

Mr. Berg. of the public are not sacrified thereby, and whether in so doing they act wisely or unwisely, fairly or unfairly between themselves, is not for the Commission to determine; the jurisdiction of the Commission is confined to inquiring whether the situation which the carriers have created is in violation of the act to regulate commerce.

"Upon complaint brought on behalf of New York City, and alleging that differentials, allowed by the defendant carriers on grain, flour and provisions from Chicago and other western points, of 2 cents to Philadelphia and 3 cents to Baltimore below the rates to New York, are unlawful under Section 3 of the Act to Regulate Commerce. Held, That the differentials are legitimately based upon the competitive relations of the carriers, that it does not appear upon the present record that the carriers have exceeded the limit within which they are free to determine for themselves, and, accordingly, that the differentials complained of do not result in unlawful preference or advantage to Philadelphia or Baltimore over the City of New York."*

It should be noted that the author says "this decision stated only that differentials were not illegal." The decision in reality declared that the existing differentials were not only not illegal, but that they did not place New York under a disadvantage.

The main reason for entering into a discussion of the paper is the fact that while the author admits that the legality of rail differentials has been adjudicated by the duly constituted public and judicial authorities, and inferentially acknowledges that the question belongs to the past and "the remedy is not with the railroads" who "have no power to protect the commerce of New York," still, there are certain inaccuracies in the facts presented, and, further, the history of rail differentials and the arguments are marshalled in such a skilful manner as to be very likely to leave the impression on a casual reader of the paper that rail differentials are founded on wrong principles, and that railroad companies engaged in the New York trade are solely responsible for the decline of the commerce of the port.

It is not for a moment contended that the author has purposely been unfair in this matter, but, no doubt, his previous legal connection with this subject has led him to give a preponderating importance to rail differentials, the main claim made in the quoted case of the New York Produce Exchange *vs.* the various railroads, and he has unconsciously allowed all other possible elements contributing to the decline of New York's commerce to be subjugated, as being unimportant factors. The purpose of this discussion is to call attention to the large number of questions and elements that should form a component part of any impartial investigation and analysis of the situation, in addition to the question of rail differentials.

In regard to rail differentials it can be stated that their legality has not only been sustained by the Interstate Commerce Commission in numerous decisions during the last twelve years, but also by decisions of the State Courts in probably every State in the Union, and of the

*7th I. C. C. Rep., 612.

FIG. 1.

Mr. Berg.

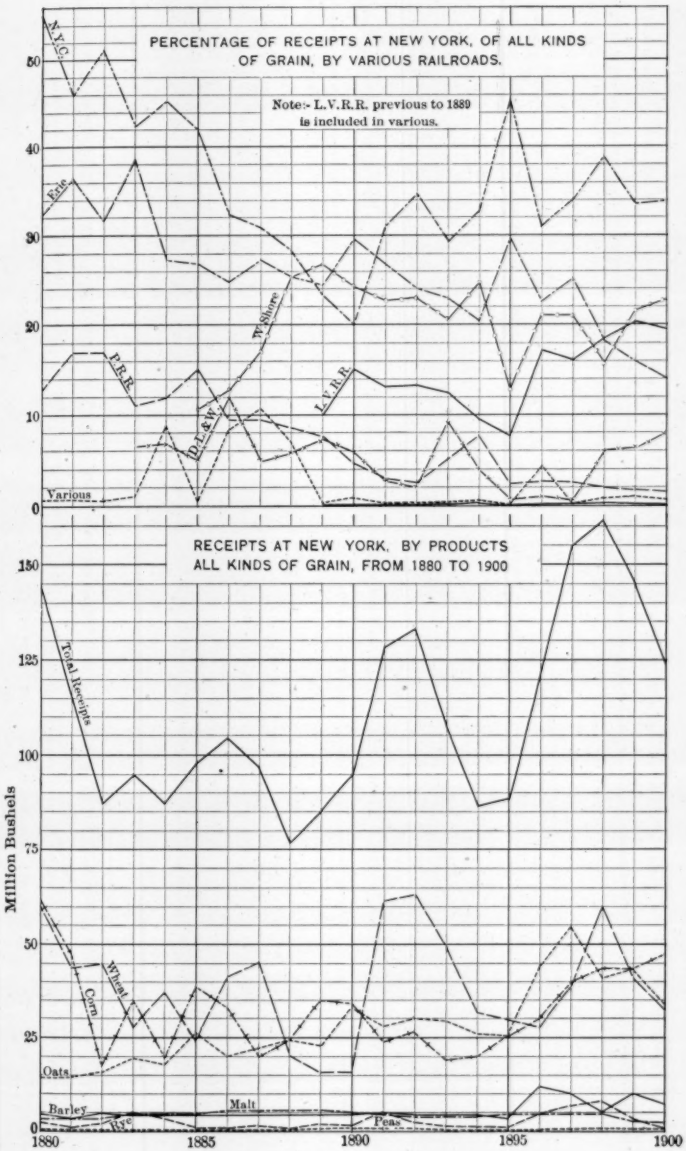

FIG. 2.

Mr. Berg. United States Supreme Court. A score of opinions could be quoted affirming the correctness of the underlying principles governing the maintenance of rail differentials.

The report of the Railroad Advisory Commission, of July 20th, 1882, mentioned by the author, consisting of Messrs. Allen G. Thurman, E. B. Washburne, and Thomas M. Cooley, appointed by the railroad companies to make an impartial investigation on railroad differential rates between the West and the Seaboard, is one of the most complete treatises on this subject, and it has been variously referred to as the "A-B-C of Rail Differentials" or the "Rail Differential Bible." Some of the conclusions reached by this Commission were as follows:

"There are influences bearing upon the charges for the transportation of property which are beyond the reach of railroad companies altogether, and which no combination among them can control. The transportation of the products and merchandise which the different sections of the country exchange with each other is only in part in railroad hands, and the carriers by rail are at all times subjected to a competition which is not only active and vigilant, but is possessed of some most important advantages. * * * No evidence has been offered before us that the existing differentials are unjust, or that they operate to the prejudice of either of the Atlantic seaport cities. Differential rates have come into existence under the operation of competitive forces; they bear some relation to relative distance and relative cost of service; they recognize as we think the relative advantages of the several seaports; and they are subordinate to the great principle which compels the carriers of property competing between the same points and offering equal facilities to their customers, to make the same rates. We therefore cannot advise their being disturbed. But we do not assume that the rates which are just to-day will be just indefinitely. They have become established by the force of circumstances, and they ought to give way if future circumstances shall be such as to render it right and proper."

It may be argued that the members of this Advisory Committee, while apparently rendering an impartial opinion, were unconsciously swayed by the railroad interests employing them. The following quotation from a decision of the Interstate Commerce Commission must certainly be accepted as coming from a body biased, if at all, in the interests of the people:

"The relative reasonableness of rates on shipments from Western points to cities on the Atlantic Seaboard is to be determined by all the circumstances and conditions that affect the traffic to the respective points between which the rates are questioned, and not solely by one standard of comparison. The length and character of the haul; the cost of the service; the volume of business; the condition of competition; the storage capacity; and the geographical situation at the different terminal points are all elements of importance bearing upon the relative reasonableness of the respective charges for transportation."*

In the New York Produce Exchange case the Interstate Commerce

* 1st I. C. C. Rep., 436.

Mr. Berg.

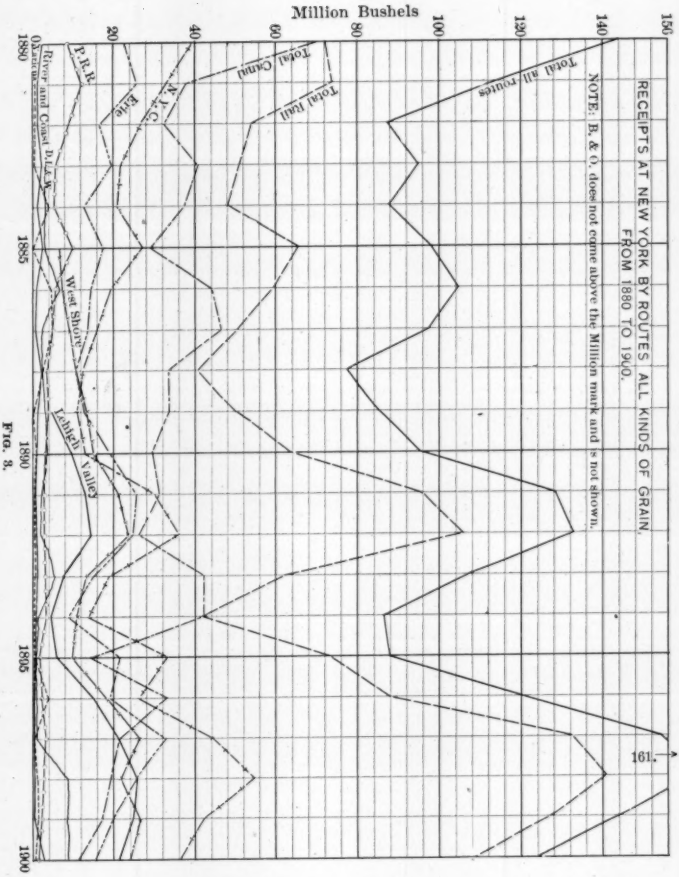
RECEIPTS AT NEW YORK BY ROUTES ALL KINDS OF GRAIN.
FROM 1880 TO 1900.
NOTE: B. & O. does not come above the Million mark and is not shown.
Million Bushels
20
40
60
80
100
120
140
150
161.
Total all routes
Total Canal
Total Rail
N. Y. C.
Erie
P. R. R.
West Shore
Lehigh Valley
1880
1885
1890
1895
1900

FIG. 8.

Mr. Berg. Commission stated: "We think, therefore, that the principle upon which these differentials are made is legitimate."

The following reference to a case decided last year by the Supreme Court of Minnesota will show the attitude of State Courts when considering methods of determining reasonable rates, and among others the making of rates in accordance with commercial conditions and circumstances. The ruling was, in substance:

"When considering the reasonableness of rates, the Interstate Commerce Commission and the Courts are justified in taking into consideration what is known as a 'commercial necessity,' namely, the application of principles when fixing rates which are forced upon common carriers by various conditions and circumstances and are in common practice among them, a business policy which actuates and influences the carriers themselves to disregard a rule of strict comparison and strict equality as between bulk, weight, or value, as well as distance of carriage. * * * This rule has constantly been recognized and acted upon by railroads, and has often been referred to and countenanced by the Interstate Commerce Commission."*

The attitude of the United States Supreme Court on the subject of variations in rates due to competitive local conditions can be inferred from the following quotation from a decision rendered last year:

"The competition may in some cases be such as, having due regard to the interests of the public and of the carrier, ought justly to have effect upon the rates, and in such cases there is no absolute rule which prevents the Interstate Commerce Commission or the Courts from taking that matter into consideration.†

Considering, therefore, that the public authorities and highest courts of the country have settled the equities in the case, and that their attitude has been well defined upon the question of rail differentials and rates governed, not merely by the distance or cost principles, but by the competitive factors and all the varying local conditions, it does not seem necessary to attempt to present long arguments as to the justice, fairness and correctness of the underlying principles. Until the fundamental law of the land is changed, railroad companies establishing rail differentials should not be indiscriminately charged with maintaining a conspiracy against the people, and be blamed for all the losses of individuals engaged in trade, or every trade grievance of commercial communities.

It does not seem fair to allow the impression to prevail that railroad companies, and they only, have been the great stumbling block in the way of maintaining the former glory of New York's commerce, unless supported by new evidence or more substantial facts than a reissue in a new form of old arguments long since adjudicated and dismissed.

It is not reasonable to assume that the half-dozen important railroads serving New York would, for practically the last twenty-five

* I. C. C., 14th Annual Report, 65.

† 175 U. S., 648

PERCENTAGES OF RECEIPTS, AT NEW YORK, BY ROUTES, OF ALL KINDS OF GRAIN, FROM 1868 TO 1900.

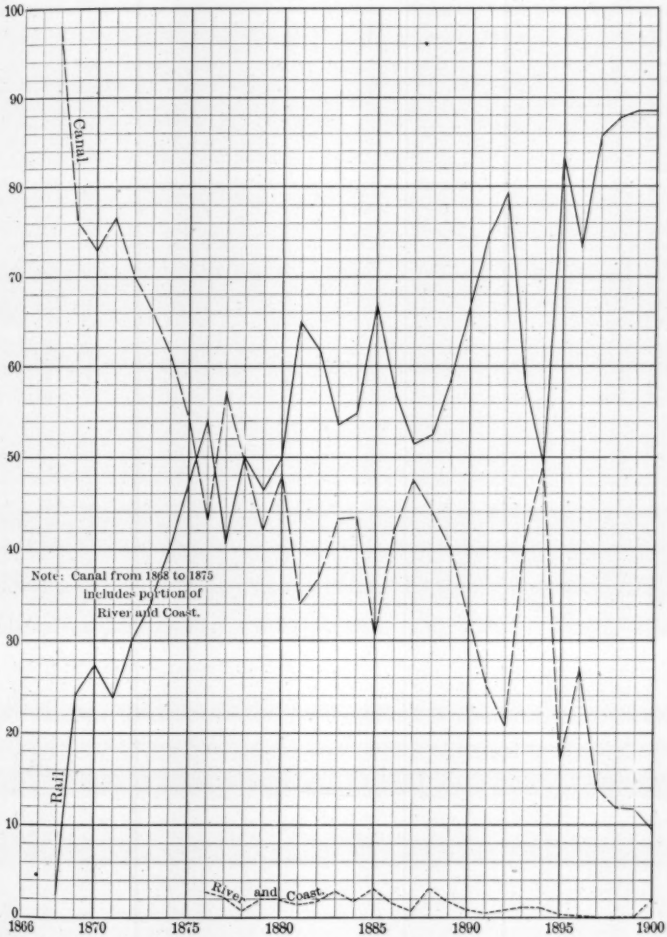

FIG. 4.

Mr. Berg. years, deliberately maintain a policy that would eventually undermine the commercial supremacy of New York, and thereby cause loss to themselves, as also to the State by the reduced taxation on their diminished earnings and value of their franchises. Even the author admits that "railroad companies do not commit suicide."

The Railroad Advisory Commission, previously mentioned, stated, in this regard:

"Railroad problems assume such different phases from year to year, and almost from day to day, that those who have authority in railroad matters may justly be expected and required to give their earnest attention and best efforts to making their franchises accomplish the great ends of equal, fair, prompt and beneficial accommodation which was intended in their grant. And those ends they should have in view in determining upon the continued existence of differential rates. Their observation of the general course of traffic from day to day and from month to month ought to enable them to determine whether the differentials are too large or too small; whether they are influencing trade unfairly and unnaturally; and whether they operate as an improper restraint upon competition; and when the improper effect is discovered, they ought to correct the wrong without hesitation or delay."

In the New York Produce Exchange case, previously mentioned, the Interstate Commerce Commission stated, in this respect:

"The lines which extend to the Port of New York are numerous, powerful and aggressive. It is difficult to believe that those lines will ever suffer any great or permanent injury to the commerce of that port, when in permitting that they must submit to a depletion of their own revenues. It might happen that some combination of these lines, for the purpose of promoting their interests at some other point, would sacrifice the Port of New York, or that for the purpose of promoting their interests as to some other kind of traffic they would sacrifice this particular traffic. If anything of that sort were apparent, if there seemed to be anything arbitrary, anything unreasonable, any undue preference against this locality or this species of traffic, it would be our duty to correct it. But there is nothing of that kind and we can do no better than to leave this matter where competition has left it."

The author states that the railroad companies have no motive for protecting the commerce of New York. The reverse is the case; they have every motive to do so. Railroads, like any other branch of commerce, are controlled by the natural laws of trade and the general law of demand and supply. They are a commercial and quasi-public body engaged in the selling of transportation, and are also promoters of production and activity in every class of trade and business. The general prosperity of the business communities they reach is immediately reflected in the railroad company's balance sheet. The two interests are inter-dependent. It is erroneous to suggest that a railroad company can so forget its true interests as to antagonize the business interests of a community it serves. Self-interest can be counted on to maintain the proper equilibrium, whether actuated by the so-called selfish motives of financiers controlling the railroad prop-

erty or the ambitious desires of the railroad officers to promote the general welfare of the property committed to their charge. Railroad officers exercise far greater care and give more thought to the satisfactory solution of complex problems connected with traffic conditions influencing the prosperity of communities reached by the road, than they receive credit for from the general public, swayed and influ- Mr. Berg.

RECEIPTS AND EXPORTS AT NEW YORK, ALL KINDS OF GRAIN, FROM 1868 TO 1900.

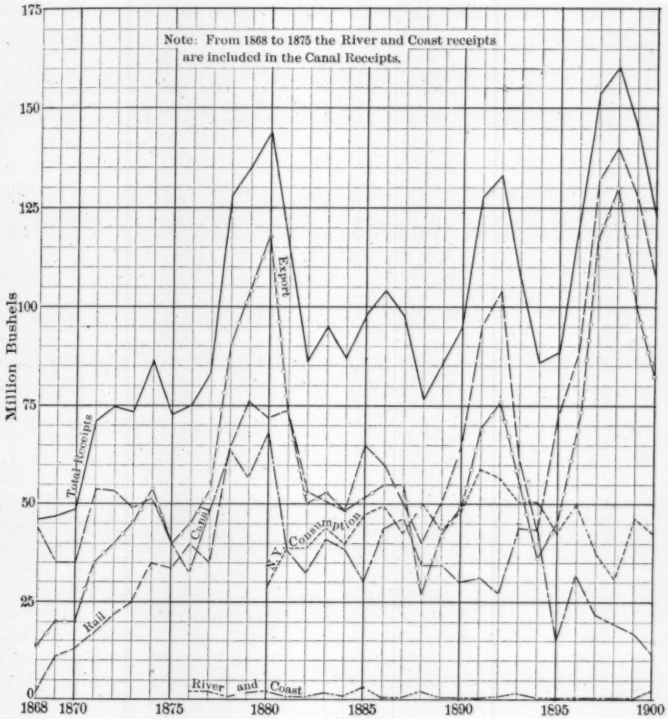

FIG. 5.

enced by the dominating ebullitions of a certain section of the daily press against all corporate activity.

If the author, in place of offering practically a wholesale arraignment of railroad companies and a general condemnation of the entire rail differential question, had entered more particularly into a discussion of all the various elements affecting the question of the decline of the commerce of New York, and had dwelt on the question of the rea-

Mr. Berg. sonableness of the amount of the existing rail differentials, he would have made a much more forcible argument.

The question of the reasonableness of the existing differentials is open to discussion and individual conclusions. What was fair and just in 1877, may not be so to-day. In 1882 the Railroad Advisory Commission reviewed the conditions and reported that the then existing differential rates, as established in 1877, viz., 2 cents and 3 cents, were fair and just, but might not be so later. The Interstate Commerce Commission reviewed the conditions in 1897, on complaint of the New York Produce Exchange, and decided that they could see no reason for making a change, as the rail differentials were not causing a disadvantage to New York. In 1897, therefore, the differentials were found to be fair, and the question is: Has any new evidence developed to change the conditions since then?

The author incidentally remarks that certain modifications in rail differentials had been made during the last year, without, however, stating specifically to what extent or in how far such modifications would affect his general arguments based upon the older conditions as applied to the future.

It is well known that the deduction of a fixed differential from a varying tariff rate becomes a matter of serious moment in proportion as the tariff rate is lessened, so that fixed differentials which would be equitable under a high tariff rate become objectionable under a low rate. The present rates on export grain from all the Eastern Lake ports are absolutely uniform to New York and its five outports, namely, for wheat, 4 cents; for corn, $3\frac{3}{4}$ cents; and for oats, 3 cents per bushel. It is obvious that with rates reduced to such low points, the question of additional deductions for differentials is not practicable.

About two years ago the railroad companies, by joint action, cut the old all-rail differentials on export grain in two, so that since then the differentials on such all-rail grain have been only 1 cent per 100 lbs. in favor of Philadelphia and $1\frac{1}{2}$ cents per 100 lbs. in favor of Baltimore, Norfolk and Newport News.

For a number of years there have been no differentials on ex-Lake grain for export, and the rates have been absolutely the same from the Eastern Lake ports, such as Buffalo, Erie and Fairport, to New York, Boston, Philadelphia and Baltimore. Ex-lake grain is practically all exported.

The conditions, therefore, during recent years, as far as the bulk of the New York grain trade is concerned, have certainly been more favorable than under the old differential rates, which were found by the Interstate Commerce Commission, as late as 1897, to be still just and not disadvantageous to New York. Yet, in spite of these reductions in the differentials during recent years, the relative percentage of New York's grain trade, compared with other ports, has been decreasing,

Mr. Berg.

U.S. GRAIN PRODUCTION, ALL KINDS OF GRAIN, FROM 1866 TO 1900.

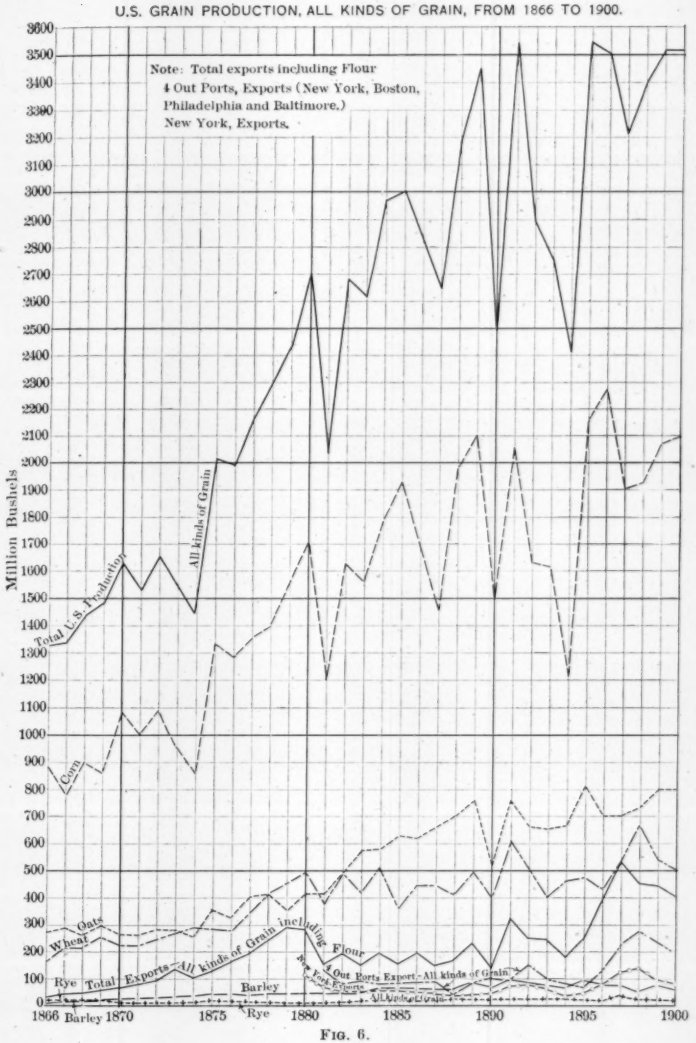

FIG. 6.

Mr. Berg. clearly indicating that rail differentials are not at the bottom of the trouble. The author has correctly stated that the development of trade takes place under natural laws and cannot be forced.

In regard to the legality and justice of making export rates lower than the domestic rates, the Interstate Commerce Commission rendered a decision in 1899 which contains the following language:

"Market conditions may justify an export rate through the Port of New York somewhat lower than the domestic rate, and Philadelphia, Baltimore, Norfolk, and Newport News usually take rates which are certain differentials below the New York rate on both domestic and export traffic. * * * Such an adjustment of rates would be to the advantage of the carrier, and just alike to the American consumer and the American producer."*

Owing to the reductions in rail differentials in recent years, many of the author's figures, when applied to existing conditions, will be cut in two, and all references to the so-called tax on ex-lake trade would have to be practically entirely disregarded to-day.

The deductions the author draws between the volume of the grain trade from 1873 to 1876, in comparison with the last four years, are ingenious, but misleading, as the conditions at that early period of the development of the railroad grain trade were entirely different from to-day, and hence do not warrant the conclusions reached, that the rail differentials are responsible for the relative decline of New York's grain trade, in comparison with other ports.

The figure, $666 000, stated by the author to represent the yearly burden upon the Port of New York due to the differentials, is grossly exaggerated, owing to errors in the assumptions. The all-rail differentials of 2 cents and 3 cents have only been in use since 1877, not since 1872; during part of the time they were not enforced, and in recent years they have been reduced in part. The ex-lake differential was only established in 1895 and was only 1 cent per 100 lbs., and has recently for a number of years been practically abolished. The author's calculation is based upon the maximum differential 3 cents on all grain, both all-rail and ex-lake, since 1872. The errors in the assumptions are apparent. After making the necessary corrections and distributing the resulting amount among half a dozen railroads, the figure is comparatively so small as to invalidate the force of the author's argument, that this contribution to the company's treasury offers "substantial reasons for the apathy of the New York roads in their efforts to protect New York commerce."

The attitude of railroads and the rail differentials will not suffice to explain the changes taking place in New York's commerce. There are many other reasons, some of which are incidentally mentioned by the author, without allowing, however, any particular weight to be attached to them.

*8th I. C. C. Rep., 214.

U.S. PRODUCTION OF WHEAT, CORN AND OATS, FROM 1866 TO 1900;
IN COMPARISON WITH EXPORTS.

Note: 10 Atlantic and Gulf Ports. { New York, Montreal, Portland, Boston, Philadelphia, Baltimore, Norfolk, Newport News, New Orleans and Galveston.

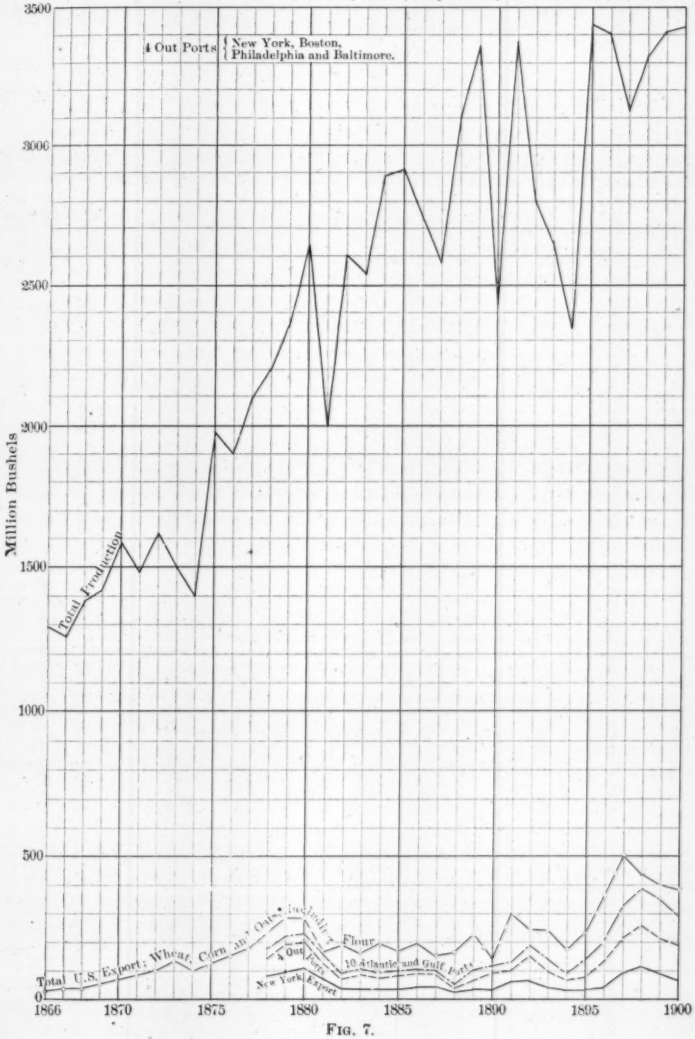

FIG. 7.

Mr. Berg. It is not feasible in this discussion to enter into a full analysis of all factors affecting the question. It has been thoroughly gone into in special reports and investigations made within recent years by the United States Industrial Commission, the United States Interstate Commerce Commission, the New York State Committee on Commerce of the Port of New York, the New York Chamber of Commerce, the New York Dock Department, the New York State Committee on Canals, and various municipal bodies and trade organizations. The several causes and effects, with reference to their comparative influence at the different ports, were discussed and investigated from almost every point of view. In addition to the relations of the railroads and rail differentials, the following features are all more or less intimately related to the subject: The decline of traffic on the Erie Canal; the improvements made by the Government, railroad and private interests in the terminal facilities at other ports; the increased business of Canadian ports, and canals and contemplated Canadian canal projects; the influence of the Gulf ports; the shifting of production and population centers; the population and domestic consumptions at seaboard cities; the shifting of the wheat and corn belts; Lake shipments; the relative rates of rail and water carriage; handling expenses; port, elevator and lighterage charges; docking facilities; ocean freights, berth and cargo-lots; vessel and steamship tonnage; regular steamship lines running from seaports; tramp steamship service; cargo-lot trade; warehousing and floating elevator interests; inspection rules, regulations and charges; brokerage and commission charges; classification systems; export and local rail rates; through rail and ocean carriage; relation of exports, imports and balance of trade; volume of manufactures at seaports; warehousing capacity and location of warehouses with reference to rail and water communication; subsidies to ocean liners; granting of free storage and other facilities; remission or reduction of charges for insurance, demurrage, etc.; guarantee of full weights; money and trade centers and banking facilities; competition of water and canal routes; comparison of water and rail rates; water navigation season and time element; traffic facilities and conditions at Chicago, Buffalo and other Lake ports, etc.

Any analysis of the general question as to the reasons for the decline of the commerce of the Port of New York must include a full consideration of practically all the foregoing subjects. The problem is one of the most complex ones that can be offered to a student of economic and transportation questions. It requires for its satisfactory solution an absolute detail knowledge of all the phases and conditions involved, and can only be answered after a thorough, systematic and intelligent examination and study from every standpoint.

The rail differentials have no doubt certain objectionable features, and the rates may not have been absolutely just and fair at all times,

EXPORTS OF ALL KINDS OF GRAIN, NEW YORK IN COMPARISON WITH BOSTON, PHILADELPHIA AND BALTIMORE, FROM 1880 TO 1900.

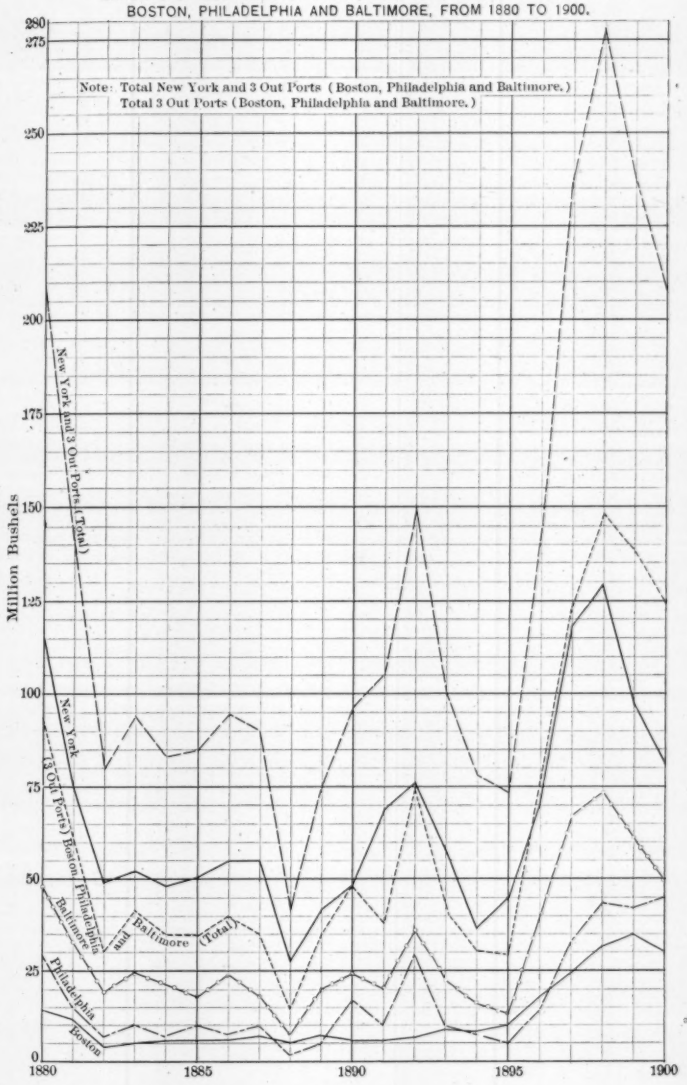

FIG. 8.

Mr. Berg. but they are not responsible for all the decline. An examination of the diagrams, showing graphically New York's percentage of exports of wheat, corn and oats, in comparison with the total exports of New York and its immediate outports and groups of ports, covering Montreal, Portland, Boston, Philadelphia, Baltimore, Norfolk, Newport News, New Orleans and Galveston, will offer the best proof that rail differentials are not causing all the trouble. New York's percentage from 1895 to 1900 has fallen in every case. The New York rail differentials have certainly no direct leading influence on shipments from the Gulf ports and from the Canadian ports, yet the same decline of New York's percentage has taken place, and at about the same ratio as at the outports, which are directly concerned. The same reduction in New York's percentage has taken place, as between New York and Boston, starting in 1891, and yet Boston is laboring under the same so-called differential burden as New York. This diagram is the best possible denial of the claim that rail differentials are solely responsible for the New York conditions. New York has lost in comparison with ports which are not at all affected by rail differentials, and in comparison with Boston, which has the same differentials as New York; clearly indicating that there are other conditions causing the general decline besides rail differentials.

The author has based his arguments as to the decline of New York's commerce largely on the decline in the grain trade of the port, hence the diagrams presented as to the New York grain receipts and exports are of value in this connection. There is no question that the proportional grain commerce of New York is gradually being reduced as compared with the total exports from its three immediate outports (Boston, Philadelphia and Baltimore), from its five outports (Boston, Philadelphia, Baltimore, Norfolk and Newport News), from the two North Atlantic or Canadian ports (Montreal and Portland), from the two Gulf ports (New Orleans and Galveston), and, further, from all the nine principal grain ports from Montreal to Galveston.

The decline is only relative, however, as the actual average yearly volume of grain receipts and exports at New York during the last five years has been greater than during any previous five-year period.

The total grain receipts at New York seem to follow a cycle of six years, and the low year occurs the second year after the high year. This rule practically holds good since 1874. The years 1892 and 1898 were the last high years. The results predicted in the New York Produce Exchange case of 1897, based mainly upon the low years of 1895 and 1896, have not been since realized, so far as the total volume of receipts at New York is concerned, the output of 1898 being the high-water mark in the New York grain trade.

New York cannot eliminate the influences at work which are causing a large part of the grain and productions of the country to seek other

Mr. Berg.

FIG. 9.

NEW YORK'S PERCENTAGE OF EXPORTS OF ALL KINDS OF GRAIN, IN COMPARISON WITH THREE OUT PORTS, BOSTON, PHILADELPHIA AND BALTIMORE, FROM 1880 TO 1900.

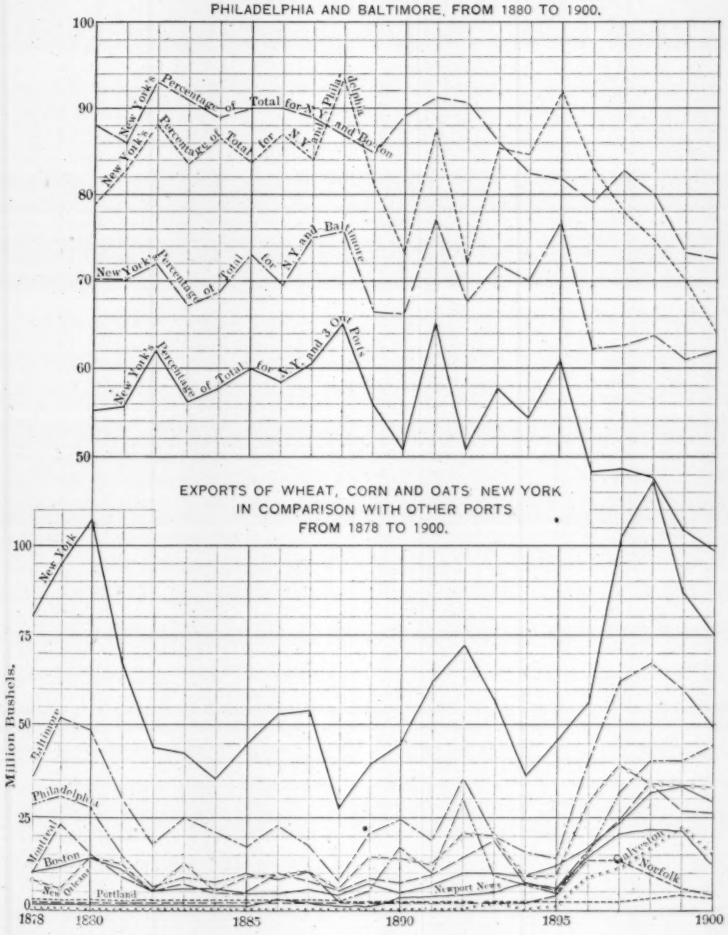

FIG. 10.

Mr. Berg. outlets, due to the shifting of the production and manufacturing centers and to new developments of railroads, canals and shipping facilities at other ports, more particularly at the Canadian, Chesapeake Bay and Gulf ports. The only successful method is for the New York railroad, shipping, warehousing and floating elevator interests, the financial and commercial bodies, and the public and State authorities to all work together for the common good. In every branch of the commerce of the port, in one way or the other, there will be found elements tributary to the relative decline of the commerce of the port, each such element being responsible for a certain percentage of the loss. Rail differentials, no doubt, have contributed a certain percentage, but in any impartial and comprehensive analysis it will be found that their percentage is small as compared with other vital features.

The author refers to excessive elevator and lighterage charges. As a matter of fact, the charges are fair if all the expenses, losses and annual charges on the investment connected with each business are duly considered. The great trouble with many comparisons and off-hand statements of unit costs is that proper allowance is not made for all the general expenses, unusual expenditures that occur from time to time, losses, and risks incurred in every business. In addition, it should be stated that, in spite of the general opinion to the contrary, railroad companies own only 20% of the storage elevator facilities of New York, and the floating elevators are under the control of a private corporation. At all other ports the bulk of the elevator service is owned by the railroad companies, and the steamship lines are more or less dependent on the railroad companies.

The author also refers to excessive grain inspection fees. This is a matter entirely under the control of the New York Produce Exchange, which body has been most pressing in its arraignment of railroad companies. Why would it not be a good scheme for the New York Produce Exchange to initiate the movement and set the pace by reducing these alleged excessive inspection fees or abolishing them altogether and absorbing the expense of the grain inspection department in their general expense account? The answer which would be given to any such proposition is obvious. But unless the non-railroad interests are willing to sacrifice something, how can they expect the railroad companies to bear all the brunt of the fight?

The general outlook for the commercial and export trade of the entire country is particularly bright at the present time, and, while New York will probably continue to lose relatively, as the facilities and developments at other ports progress, she will, no doubt, always retain a large volume of the total business. To stop or reduce this relative decline and to increase the total actual business should be the endeavor of all the parties interested. This should not be done, however, by aiming to throw the odium on some one else, or selecting

EXPORTS OF WHEAT, CORN AND OATS, NEW YORK IN COMPARISON WITH GROUPS OF PORTS FROM 1878 TO 1900.

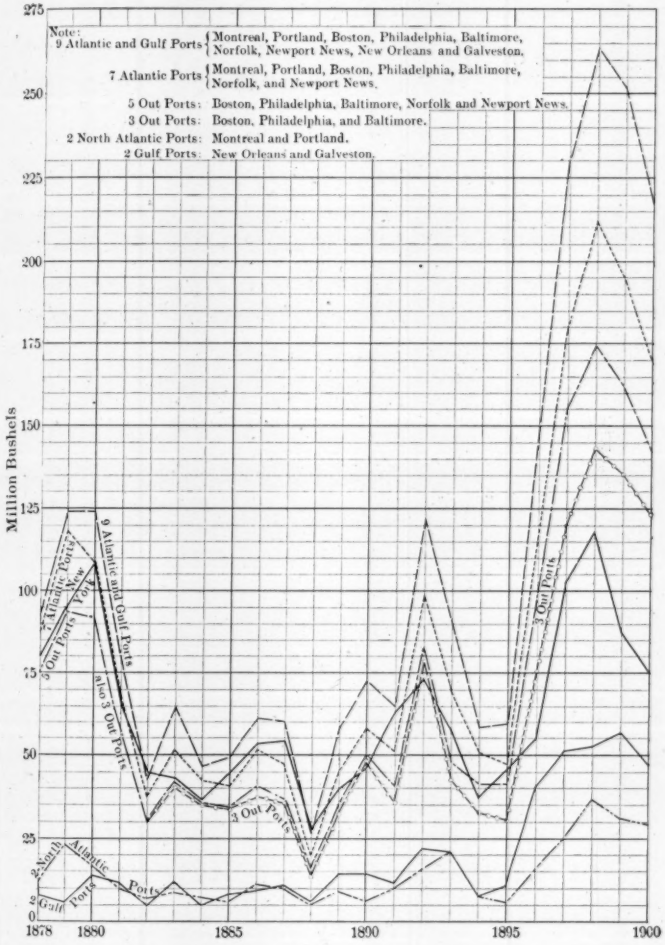

FIG. 11.

Mr. Berg. some new scheme as offering the only salvation, but by united action and concessions all around. The railroad companies have cut in two the old all-rail differentials on the bulk of the grain shipments, and practically abolished ex-lake differentials, and are undoubtedly willing and endeavoring to solve the many problems involved, to the best advantage of the commerce of the port. In spite of the author's statement to the contrary, it is perfectly evident that railroads have a powerful motive to foster the commerce of the port. Even if it is largely due to self-interest, it exists. They are bound to protect themselves, owing to the enormous investments already made in terminal facilities at the port. Every new scheme and railroad improvement should be encouraged by the public authorities and commercial interests, as it offers an additional guarantee that the railroads must do their part in the general forward movement. The question is: Will the other interests involved do their share?

In conclusion, it will be pertinent to remark, in connection with the second section of the paper, that any enlarged canal scheme will, no doubt, contribute its share to the general improvement in the commerce of the port, but it should not be considered as the panacea for all ills.

The author claims that the railroads have been practically the cause of all the trouble, and that they cannot be looked to for a remedy, as they are powerless to act, even if they had sufficient motive and were willing to protect the commerce of the port. The aim of this discussion has been to refute the view that rail differentials are the sole cause of the decline, and to emphasize that railroad companies are not apathetic, but have every motive to assist in protecting their own interests as well as those of the entire community. Further, that the problem is a very complex one, involving many other interests besides those directly controlled by the railroad companies.

The solution will not be advanced by vituperations and one-sided arguments throwing all the blame on one interest, but requires impartial investigation and united action and concessions, not only by the railroad companies, but by all commercial, shipping and harbor interests, trade organizations and public authorities. The efforts of these various non-railroad interests, heretofore, have been largely directed to a general arraignment of railroad companies and seeking to establish a new so-called competitive weapon with which to fight the railroads. But there is no cause for such an attitude. Whatever will assist the general prosperity of the port will, by reflex action, aid the railroads. Increased exports will bring increased imports and manufacturing activity, all of which will contribute to increased railroad earnings.

An enlarged canal will undoubtedly be of benefit to the port and cause the commerce of the Great Lakes to seek an outlet through the

Port of New York, thereby increasing the commerce of the port and the general prosperity of the community. The railroad companies will not only benefit by the increased traffic thereby caused to and from New York, but will be aided in successfully resisting the inroads now being made and to be expected in the future from the Canadian

Mr. Berg.

NEW YORK'S PERCENTAGE OF EXPORTS OF WHEAT, CORN AND OATS, IN COMPARISON WITH VARIOUS GROUPS OF PORTS, ETC., FROM 1878 TO 1900.

9 Ports:- Montreal, Portland, Boston, Philadelphia, Baltimore, Norfolk, Newport News, New Orleans, and Galveston.
7 " " " " " " and Newport News.
5 " Boston, Philadelphia, Baltimore, Norfolk, and Newport News.
3 " " " "
2 Gulf Ports:- New Orleans, and Galveston.
2 North Atlantic Ports:- Montreal and Portland.

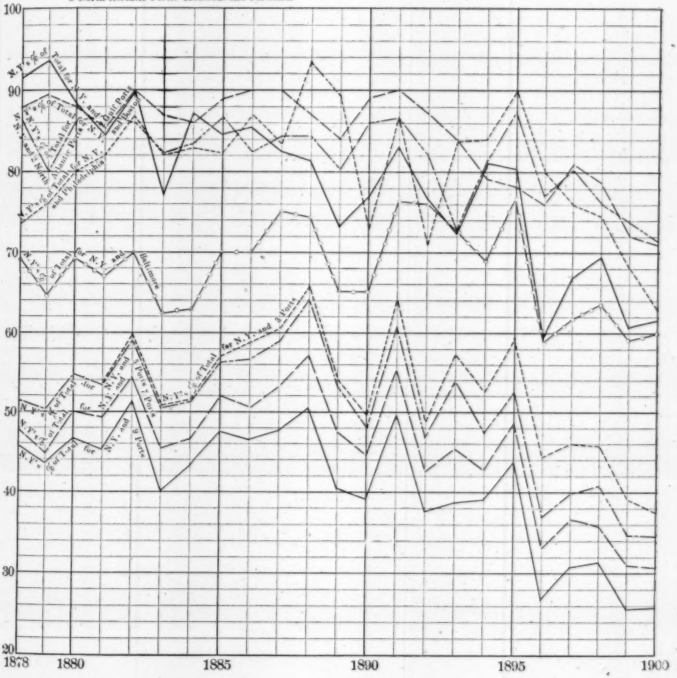

FIG. 12.

canal, railroad and terminal improvements, as also the tendency for the products of the Northwest to seek southern outlets. As the author has correctly stated, conservatism and force of habit in trade cause traffic to follow certain well-established channels and outlets, and, it

Mr. Berg. may be added, along the lines of least resistance. Create these conditions, in any way that may seem best, so that New York will become the natural outlet for the rapidly increasing traffic from the Lakes and the West, and it will be found that railroad companies and all parties affected, including the canal interests, will benefit in the general prosperity. The author has stated correctly that conditions must be created to allow the development of commerce under the natural laws of trade. The truth of this generality should be borne in mind by all interests concerned in the prosperity of the port, and their efforts directed to a general improvement in all branches of commercial activity, in place of fostering animosity against railroads, if the decline in the commerce of the port is to be halted and the glorious position of New York as the chief port of the country maintained.

Mr. North. EDWARD P. NORTH, M. Am. Soc. C. E.—It will be noticed that the yearly amount of the differentials mentioned by Mr. Blackmar, viz., $666 000, is treated from a local point of view, as if solely taken from the net resources of the inhabitants of the City of New York. But it is apparent that by forcing freightage into inconvenient and more expensive channels, the differentials have compelled the producers of the West to contribute their full amount, not only on all produce reaching the Atlantic seaboard, but, so far as values at the point of production are governed by the net receipts at the place of consumption, the entire product has had its value to the producer decreased *pro rata* by the differentials imposed.

While it is now impracticable to predict the effect of the comparatively new community of interests between the railroads on the present practice of imposing differentials on the traffic of the country in discrimination against the cheapest route, there seems to be no ground for expecting its discontinuance. And the only alleviation of the impost that presents any certainty of automatic action seems to lie in a competing public waterway capable of passing the largest vessels navigating the Great Lakes.

Mr. Green. ANDREW H. GREEN, Esq. (by letter).—This subject ought to command the attention of the city authorities and of such commercial and financial organizations as have the welfare of the City of New York at heart.

The differential rates that the railroads have taken the liberty to establish, and that work so disadvantageously upon the interests of the city, ought to be abolished, and New York enter again upon the advantages with which Nature has provided her.

The reasons that were urged for the establishment of these rates have largely ceased to exist, and prompt means should be taken for their abolition.

It is stated by a high authority that:

"An investigation of the relation of differentials to the rates

Mr. Green.

charged in 1877 and in 1897 shows that by the decline in the rates, which has taken place during that period, the differentials are now even more effective against New York than when first established. The rate on wheat by rail from Chicago to New York has fallen about 40% since 1877. At that time the percentage of the differentials allowed Philadelphia and Baltimore were, respectively, 10% and 18% of the rate on wheat charged from Chicago to these ports. In 1897, by reason of the decline in the rate on wheat, the percentage of the differentials had increased to 19% of the rate to Philadelphia and 31% of the rate to Baltimore. Thus, by the continuous application of differentials, originally established to offset conditions existing twenty years ago, the beneficial effect to ports south of New York has increased in proportion to the decline in rates, and it has been still further augmented as New York has been dispossessed of the advantages she then enjoyed. * * * It is true that the influence exerted by the differential rates is entirely disproportionate to the conditions now existing, and therefore harmful to the interests of New York."

It is a strange condition that railroad corporations that owe their existence and their right to conduct their business in this State, should be permitted to use their powers to render nugatory the natural advantages that the State possesses. It is a clear case of matricide.

Mr. Wisner.

GEORGE Y. WISNER, M. Am. Soc. C. E. (by letter).—Mr. Blackmar has given in his admirable paper the correct diagnosis of the evil arising from railroad freight differentials, and has prescribed the proper remedy to restore normal conditions in transportation business, but there is some question whether the homœopathic dose which he prescribes will be sufficient to completely eradicate the germs of the evil, which, unless held in check, are certain to kill the traffic on all water routes, from the Lakes to the Atlantic, which cannot furnish better transportation facilities than existing lines.

The differential agreement, in connection with railroad ownership of the steamship freight lines on the Lakes, is one of the most ingenious arrangements for controlling the routes that the traffic between the Lake region and the seaboard must follow which it is possible to devise.

Under this arrangement the railroads best located for furnishing cheap transportation have retained their full share of the transportation business, while other roads, where the natural conditions for obtaining traffic are less favorable, have developed a large increase in business at the expense of the traffic of the Erie Canal.

The railroad differential agreement of 1877 became fully effective about 1880, since which date the traffic of the Erie Canal has decreased 50%, while that of the railroads has been trebled.

When we consider that this result has been accomplished with railroad rates nearly double those of the Erie Canal, it is quite evident that the railroad freight differentials alone have not been the only influences which have diverted the commerce of the Erie Canal to the railroad lines between the Lake and Atlantic ports.

Mr. Wisner.

The grain and package freight on the Lakes is transported in steamers running in connection with the railroad lines from the Lakes to the seaboard, over which lines through bills of lading are given from the Lake ports to the ocean terminal.

The Chicago shipper can turn his elevator receipts into the bank with each shipment, and get returns at once with which to continue business, but with freight shipped by the Lakes and the Erie Canal, no such arrangement is possible, and even with the 12-ft. barge canal, which Mr. Blackmar advocates as a regulator of railroad combinations, it is very doubtful whether existing conditions would be much improved.

It is generally admitted that such a waterway would necessitate transfer of freight at Buffalo, and, without the formation of large transportation companies controlling freight steamers on the Lakes and lines of barges on the canal, the railroad and Lake steamship lines would still be able to control the movement of freight the same as under existing conditions.

It has been stated that transportation companies would be formed and barges built for the route by the time that the waterway could be completed. This would be true, probably, if there did not exist the element of danger that a cheaper transportation route from the Lakes to the Atlantic will be constructed long before a 12-ft. barge canal could be completed between Buffalo and the Hudson River.

Capitalists will not expend large amounts in the construction of fleets of barges unless there is a fair chance that the expected traffic will not be diverted by the building of other waterways better adapted for the business, and over which the commerce of the Lakes can be transported to the seaboard for a much lower rate.

It is safe to say that if the fact should be established that the waterway from Lake Erie to the Hudson is to have a depth of only 12 ft., the construction of a 21-ft. ship canal from Georgian Bay to Montreal, *via* the French and Ottawa Rivers, would be commenced within a year.

Over this route freight could be carried from Chicago to Montreal for about the same cost as from Chicago to Buffalo, a condition which would destroy all chance of developing traffic on a barge canal from Buffalo to the Hudson, except for the distribution of products for home consumption.

If, instead of a 12-ft. barge canal, a waterway of sufficient dimensions be constructed to allow the passage of the best modern Lake freight carriers, it would be impossible for any railroad combination to compete with any better success than with the present steamship transportation on the Great Lakes.

With the facilities for transportation which now exist, and which, in the opinion of the writer, will not be materially improved by the construction of a 12-ft. barge canal, the railroad companies have no

motive for changing the differential freight arrangement, and, as Mr. Blackmar has clearly shown, could not discontinue it, even if to the interest of some of the companies to do so. Mr. Wisner.

The necessary action to correct the unjust discrimination from railroad freight differentials involves, to a large extent, a correct interpretation of the problems discussed recently in the papers on Canals between the Lakes and New York.*

That discussion pretty thoroughly established the fact that any waterway requiring transfer of freight at Buffalo would not prevent or satisfactorily modify the evils of existing transportation arrangements, but, in regard to the maximum limit of dimensions of waterways to produce the most beneficial results, there seems to be a diversity of opinion which it is difficult to reconcile with the fact that most of the elements necessary for the solution of the problems are fairly well determined.

This, however, is largely due to some of the conclusions being based upon assumed data instead of upon well-established facts relative to the volume and transport rates of Lake and ocean commerce, and upon the cost to maintain waterways of different dimensions through the Lakes and from the Lakes to the Atlantic.

Mr. Blackmar states that, "according to the report of the Roosevelt Commission on Canals, 60 000 000 tons will not over-estimate the freight traffic on the Lakes in 1898." This amount is probably 50% in excess of the actual amount of freight carried on the Lakes in 1898.

The table of tonnage given in the report of the Commission on Canals, with a few exceptions, is compiled from the statistics of the entrances and clearances at the different Lake ports, and includes the tonnage of passenger steamers and of freight carriers entering and leaving ports without any cargo whatever, thus making the volume of registered tonnage largely in excess of actual freight carried.

L. E. Cooley, M. Am. Soc. C. E., wrote a discussion on the papers on "Canals between the Lakes and New York,"† which was not published in the *Proceedings*, and was not available at the time that the writer submitted his final discussion of those papers, and did not receive the consideration which should have been given it.

The conclusions reached by Mr. Cooley are apparently based upon assumptions which should be carefully considered before being given very much weight.

The dependence of the cost of transport upon the capacity of the freight carrier, the speed in restricted channels, the length of haul, the cost to construct and maintain the waterway, the average detention at terminals, and the actual expenses of conducting the transportation

* *Transactions*, Am. Soc. C. E., Vol. xlv, June, 1901.

† *Transactions*, Am. Soc. C. E., Vol. xlv, June, 1901, p. 303.

Mr. Wisner.

business, are such that any material modification of these factors will make a radical difference in the resulting relative values of the different systems and methods of transportation. All these elements are now sufficiently well established, and there should not be any legitimate reason for any great variation in the values used.

Mr. Cooley, in his discussion, assumes certain relative amounts for the cost of transport on railroads, ocean steamers, lake steamers and barges, which the writer is unable to reconcile with the rates reported in commercial and Government reports. He assumes the rates for actual ocean carriage at $\frac{1}{5}$ mill per ton-mile, and for Lake and 21-ft. waterway, at $1\frac{3}{4}$ times the ocean rate.

The rates for ocean freights (New York to Liverpool), given in the report of the New York Produce Exchange and in the report of the Committee on Canals of New York State, varies from 0.6 mill per ton-mile for wheat to 1.4 mills for flour and provisions, or an average of 1.0 mill per ton-mile. Similar freight from Chicago to Buffalo varies from $0.50 to $1.00 per ton, or an average of about 0.8 mill per ton-mile, including terminal expenses.

By using the actual cost of maintaining and running modern freight carriers, and adding the fixed charges for the cost, maintenance and operation of a 21-ft. waterway, it can be shown that package freight can be carried from Chicago to New York with a fair profit to the shipowner for less than 1.0 mill per ton-mile, and since the ocean rate for similar freight is fully as large, it is difficult to conceive why the Lake and waterway rate should be assumed at $1\frac{3}{4}$ times that of an ocean carrier on the ocean and through a 30-ft. waterway to the Lake ports.

Mr. Cooley states that his rate of $\frac{1}{5}$ mill per ton-mile for actual ocean carriage corresponds to $1.20 per ton between New York and Liverpool, and 0.79\frac{1}{4}$ between Chicago and Buffalo by Lake, but, since the actual average rate from New York to Liverpool is reported at $3 per ton, Mr. Cooley should give some authority for the rates on which his conclusions are based, which differ so widely from the rates quoted in the commercial and Government reports of domestic and foreign trade.

Mr. Cooley omits the fixed charges for deepening and maintaining Lake harbors for 30-ft. navigation, which would add over $1 000 000 to the annual expense account; and he assumes that the "over-ocean movement," using the waterway, would amount to 24 000 000 tons annually. The capacity of a single-lock canal would not be much more than this assumed foreign movement, and, since the domestic traffic on the waterway would be fully as large as the foreign freight, the fixed charges should be increased for the cost to construct and maintain a duplicate system of locks, or else the estimated movement on deep-draught ocean steamers through the canal should be greatly diminished.

Since products for domestic distribution can be transported more cheaply in freight carriers adapted to the Lake and waterway routes than in an ocean steamship, the domestic traffic should not be used as a part of the over-sea movement. Mr. Wisner.

It is true that the economical freight carriers of the Lakes could use the 30-ft. waterway, and make a little better speed than in a 21-ft. waterway, but the saving in time will by no means warrant the expenditures, and, therefore, in estimating the advantages of the deeper waterways, only such commerce should be considered as will be benefited by the increased facilities.

With the rates which have been established by years of close competition on both the ocean and the Lakes, and the accurate estimates of cost to construct and maintain waterways of different dimensions from the Lakes to the Atlantic, based on the extensive surveys and investigations made by the General Government and the State of New York, there seems to be no legitimate reason for assuming data, in the discussion of rates, which are at variance with these determined values, and if such established rates and estimates are used by all investigators as fully as the conditions will permit, the resulting comparative values of proposed water routes should not differ materially.

WILLIAM G. RAYMOND, M. Am. Soc. C. E. (by letter).—In order to discuss this paper properly one should, perhaps, be familiar with all the details of differentials, for the differential is the burden under which it is supposed New York City is laboring. Mr. Raymond.

Again, one should perhaps be familiar with the possibilities of rail and water transportation, both as to cost, tariff and shipping methods. It would be absolutely necessary for one to be versed in these and to discuss the subject in detail were it not that Mr. Blackmar fails entirely to prove a case. It is left so that even a layman may see its faults, and a layman will attempt to show some of them.

Mr. Blackmar's contentions, neither of which is substantiated, are these:

1. Railroad differentials have resulted in sending to other Atlantic ports traffic properly tributary to New York; this is an evil needing a remedy.

2. An adequate canal across New York State is the only agent that can secure the one remedy, namely, the creation of "such conditions that the natural operation of the laws of trade prohibit the discrimination" of the railroads.

In these contentions Mr. Blackmar simply sets up a wooden man and proceeds to knock it down; he does not even set his effigy firmly on its feet.

In order to make this case clear the procedure should have been somewhat as follows:

1. It should be shown what traffic is properly tributary to New

Mr. Raymond. York; that some of this traffic has been diverted elsewhere; that this diversion is the result of some interference with the "natural operation of the laws of trade," and that this interference is the rail differential.

2. A remedy could then be suggested, and, if possible, demonstrated beyond reasonable doubt to be a cure and to insure the "natural operation of the laws of trade."

Probably Mr. Blackmar supposes he has accomplished these results, but an endeavor will be made to show that he has not proved that any traffic is properly tributary to New York; that he has not shown the diversion of any traffic from New York; that he has not shown any interference with the "natural operation of the laws of trade," and that he has not shown even that the rail differential is wholly or largely responsible for the lessened percentage of traffic received at the Port of New York, though he does show the fact that New York now receives a smaller percentage of grain and flour than formerly, when the Erie Canal—an artificial highway—was the only practicable route from the West to the Atlantic seaboard.

Traffic received at any port may be divided into two classes: That for local consumption and distribution to the surrounding territory; and that for export. The first class is certainly naturally tributary to the port; the second class is naturally tributary only in so far and so long as the port is a point on the line of least resistance from the field or door of the shipper to the door of the consignee.

Imagine a long water-pipe leading from a pump to some point of discharge and open only at the end. All the water pumped in goes to the end of the pipe and if that end is in New York, New York gets 100% of the water pumped. Let now a small lateral leading to Philadelphia be introduced into the pipe; no matter how small it is some water will go through it, and New York will no longer get 100% of the water pumped.

As laterals multiply, to Boston, to Baltimore, and to Newport News, New York will receive a smaller and smaller percentage of all the water pumped. As the demand for water increases the pumps are worked harder, new ones are added, each locality receives more water than before, and this may help the percentage of New York unless at the same time Philadelphia's demand has outgrown the capacity of her lateral and a larger one is laid.

Imperfect as is the analogy, it will, perhaps, help to emphasize the fact that as other ports grow in importance and increase their facilities for receiving and shipping export traffic, that port which originally, because of some great advantage, controlled practically all the business must expect to lose in relative amount—and should be well satisfied if it maintains its position as the principal port of the land.

A port may be on a line of least resistance for export traffic because

of its peculiar geographical position; because of the excellence of its service by inland and water transportation lines; or for both reasons. New York has the advantage of geographical position, and excellence of water service. Water service depends on the accessibility of the port, the character of its harbor, and its dockage and storage facilities; in these items New York may always have the advantage of all other American ports. Mr. Raymond.

As to inland service: Before there were railroads of much efficiency, New York had, in the Erie Canal, a distinct advantage over other ports. As railroads multiplied in number and efficiency the Erie Canal became less and less a factor, so that it may be fairly said that to-day New York is not better served in proportion to her needs than are several other Atlantic ports.

However, there are no ports that ever will or can have the advantage of New York in inland service, and since none can ever hope to equal New York in water service, so New York must always be the chief American port. But New York cannot hope to always control all the export traffic. If it could there would be no other ports.

As other ports increase in number and in population and excellence of service both by water and rail, just so surely will they secure increasing quantities of export traffic to load the incoming vessels, and just so surely must New York lose in percentage of the whole export traffic, though losing nothing, but rather gaining, in absolute volume. The large percentage of the export traffic which was once naturally tributary to New York is no longer naturally tributary to that port. Improved rail transportation and competition have in various ways made it easier for some traffic to go by other lines.

One of these ways may be the rail differential. To attribute New York's whole loss in relative traffic to this cause is absurd. It has been due to the perfectly "natural operation of the laws of trade" to increase in corn production and the facilities for getting it to its natural market, a southern port; to the increase in importance of the Pennsylvania iron industry naturally tributary to Philadelphia or Baltimore; to the change in the center of the milling business, by which the greater portion of flour is made nearer Baltimore than New York; and to many other causes that cannot be specified.

Corn is a southern and southwestern product, and that it should ever have found its way in great quantities to New York must have been due to lack of proper facilities for transporting it to and marketing it at the southern ports to which it is naturally tributary because of its geographical position.

There are probably no data available that will tell to what extent the differential is responsible for the diversion of traffic from New York, but there are some figures which may give a little indication as to what this effect has been.

Mr. Raymond. Mr. Blackmar states that the lake and rail differential is not only a discrimination against New York in favor of Philadelphia and Baltimore, but it is a discrimination against Buffalo in favor of Fairport and Erie.

Let the effect of this discrimination against Buffalo be looked into.

Grain is about the only item that can be considered, because it is the only very large item of export traffic originating along or beyond the Lakes. Almost every one of the many Lake ports receives some ex-lake grain, either for local distribution or transhipment, and hence Buffalo cannot expect to be the only Lake port receiving such grain. What may be its fair share of such traffic may not be determined, but in 1900 it received about 86% of the ex-lake grain, while Erie and Fairport combined received about 6.2% of the traffic. Does this look as if the differentials had seriously hurt Buffalo, even assuming, what is probably not true, that the differential is responsible for all the grain received at these two newer ports. Is the diversion of this portion of the Lake export traffic sufficient cause for any costly canal construction? But it is asserted that the greater part of this ex-lake differential has been abolished.

Again, Mr. Blackmar states that for the four years preceding the differential agreement New York secured 57.2% of all the flour and grain received at six Atlantic ports, while in the four years last passed New York's percentage was 36.9, and a similar statement is made concerning exports. Why is this sort of comparison made? Why compare conditions in 1875 with conditions twenty-five years later, and attribute the whole change to the operation of the differential?

Has there been no legitimate progress in the other ports in twenty-five years? If the differential alone produced this effect, it should have accomplished it in much less than twenty-five years—notwithstanding "conservative tendencies," which are notoriously of little effect in America.

Is it hoped by the construction of a costly canal to re-divert that traffic which has become naturally tributary to these other ports?

If this could be done, and various recent articles in *The Railroad Gazette* and *Engineering News* have shown it to be at least problematical, and should be done at the expense of New York State, these other ports would perhaps have very little to say, but would insist on still larger differentials to offset the unnatural advantage secured by New York. But if it were done at national charge, would not these same other ports have a just grievance?

And not only the other ports, but the transportation companies. It is the settled policy in this country to leave the transportation business to private interests, Government reserving only such control as shall prevent unjust discrimination in charges. Under this policy,

great railroads and great steamship lines have come into existence. Government has done much for the steamship companies in the way of improving harbors and channels; and, in helping navigation, Government has in some measure indirectly assisted the railroads. But for Government to spend $300 000 000 or any other sum in building an artificial waterway for the free use of the owners of water craft in direct competition with established railroad lines, or, if a toll is charged, lending Government credit to such companies for the building of their permanent way, would be a commercial crime; and it would be a double crime for Government to provide the necessary money by any form of taxation that would compel the railroad companies to pay a portion of the cost. Mr. Raymond.

But this is a digression; we will return to Mr. Blackmar's figures.

As has been already stated, corn is certainly naturally tributary to southern rather than northern ports, but—including corn—the receipts of grain at New York in 1900 were about 41½% of the total receipts at the six Atlantic ports mentioned, the receipt of flour about 36½% and the receipt of grain at New York twice the receipts of any other port. Now, excluding corn, the receipts of grain at New York in 1900 were about 53½% of the total receipts at the six ports mentioned, and about three times the receipts of any other port. This does not look like a diversion of magnitude due to the differential.

Such different conditions govern the shipment of the various kinds of grain and grain products that it is manifestly improper to lump them in such a discussion as this. The truth is, that, considering these same six ports, New York has lost in relative receipts about 50% of flour, 21% of wheat and 44% of corn. A considerable portion of the loss in wheat has been lost to Boston rather than to southern ports. New York still secures over 50% of the wheat and oats—naturally northern products—while New York and Boston together receive over 71% of these products.

Taking into account the natural operations of the laws of trade during twenty-five years, it is not probable that the rail differential has worked much harm to the Port of New York.

This brief exposition finds support in Mr. Blackmar's own argument, for he has furnished good ground for doubt as to the soundness of his position when he tells us in the opening paragraphs of his paper that the Interstate Commerce Commission has looked very thoroughly into this matter, and has decided that such differentials as exist constitute neither unjust nor unreasonable discrimination against the Port of New York.

Mr. Blackmar, in speaking of the east-bound Lake traffic, says:

"The greater part of this traffic is in the coarser products, and the natural destination is the Port of Buffalo, * * * and the natural outlet is the Erie Canal."

Mr. Raymond. This is mere assertion, and may be met by the simple fact that the Erie Canal cannot carry the freight, and the railroads can and do carry it at a less total cost than the canal imposes.

Why, then, is the natural outlet the Erie Canal? It is fair to say that the natural outlet will be the Erie Canal when that canal is made better than the railroad, which can be shown to be impossible at a cost commensurate with the value of the improvement.

Mr. Blackmar, after insisting, in spite of the assertion of the Interstate Commerce Commission to the contrary, that the differentials are wrong, and work an unjust discrimination against New York City, Buffalo and other cities in New York State, asks: "Is there any remedy, and if so, what?"

It is really not clear that Mr. Blackmar has discovered any disease that needs a remedy. In a country as great as the United States, with as many miles of sea coast, and as many good harbors, there would indeed be indication of disease if one port should forever maintain a given relative supremacy in traffic in one commodity, to say nothing of all commodities, over all other ports. Somewhere there would be not only illness, but death.

The need for a remedy for the ills imagined by Mr. Blackmar is doubtful, but it may be well to consider one or two errors that develop in the course of his argument. It is stated that the tax, the differential, does not apply to that portion of grain brought to New York by canal. It is not admitted now that any portion of grain received at New York pays the differential over and above a fair freight rate; but granted for the argument that locally-consumed rail-carried grain does pay such a tax over and above a fair rate, it follows that locally-consumed canal grain pays precisely the same tax; for it has long been well known by all persons not wilfully or accidentally blind to the real situation, that the railroad makes the rate, and the canal boatman goes enough under to get some business. The railroad has been the regulator of rates, has forced the canal almost out of business, and to-day the canal carries only such grain as the railroad concedes to it. The differential exists in the canal rate just as certainly as it exists in the rail rate.

Again, speaking of the enlarged navigable waterway between Lake Erie and the Hudson River as the one remedy that is sure and effective, Mr. Blackmar says: "This agency can never make agreements with railroads to divert traffic to any other ports," etc., etc.

The control of the traffic of a canal of limited capacity is in these days a simple matter, and just so surely as such a waterway is built to work harm to established railroads, just so surely will the railroads control it, and make agreements that will protect themselves.

Again, Mr. Blackmar states that it has never been claimed that

rail rates of 2.5 mills per ton-mile have been profitable to the railroads, except to aid in the destruction of the canal. Mr. Raymond.

This is quite contrary to the fact. Such claims have been made, and it may be easily demonstrated that such rates are profitable in themselves.

But rates have always been, and will always be, fixed by what the traffic will bear; which means, when intelligently interpreted, fixing such a rate that the net return to the transportation company will be a maximum. And this applies to water rates as well as to rail rates.

From present indications it will probably be but a short time, if it is not now, before the Lake traffic will be controlled by the railroads.

If the Great Lakes can be so controlled, how long will a canal across New York State be a potent factor in controlling freight rates to a degree that will in anywise compensate for the enormous outlay involved?

The truth is that the Erie Canal has come to be a great political power, and neither party in the State dares oppose its continuance. Meanwhile the conviction is rapidly spreading that it is a "hindrance to the commercial prosperity of the State," and it is probably true that within a few years the party that advocates the abolition of the canal, which imposes a tax on the many for some small benefit to a few; which, more than any one agency, is instrumental in degrading New York State politics; and which, in spite of assertions to the contrary, can be shown to have cost the State many millions of dollars more than the State has received from it, though it is granted that it has served a good purpose in the past; this party will be placed in control of the State by an overwhelming majority.

Summing up what it is hoped has been at least suggested:

1. Mr. Blackmar has not shown what traffic is properly tributary to New York; that any such traffic has been diverted by unnatural or improper methods; that the Lake and rail differential has injured Buffalo to any serious extent; that, indeed, any ailment exists requiring heroic measures by State or Nation to cure.

2. Mr. Blackmar has by no means shown that the agency he suggests will accomplish the result he desires.

HERMAN CONROW, Jun. Am. Soc. C. E. (by letter).—A great deal of light has been thrown upon the merits of the various schemes of water communication between the Lakes and New York by the papers and discussions of the last three years. There are still some phases of the question, however, which have not been discussed. Mr. Conrow.

So much has been written concerning the "decline of commerce at the Port of New York" that by many it is apparently accepted without question as a fact. Because formerly 70% of the export trade of the United States cleared from that port, it is claimed that as this high per-

Mr. Conrow. centage has not been maintained, necessarily the foreign commerce has declined. But the idea that the Port of New York should at the present time and for all future time export 70% or any fixed percentage of the products of this country is a gross reflection upon the size and resources of the United States.

The rate of growth of the commerce of New York is a very difficult thing to determine. There are two classes of statistics which are readily accessible, viz., the values of exports and imports and the net registered tonnage of vessels entering and clearing from the port. There are no statistics which show the tonnage of commerce entering and clearing from the ports of the United States, except as it may in part be obtained from the values and quantities given. It is obvious that statistics based upon values alone cannot be depended upon to show either growth or decline, for the commerce of the Port of New York might consist of exports of gold and imports of diamonds—swelling the values to enormous figures—yet the entire commerce use but a single ship. In this case it could not properly be claimed that the commerce of the port was growing. Nor can statistics of the net tonnage of vessels entering or clearing be depended upon to show anything more than the general direction of growth or decline; and even when values and tonnage are taken in conjunction, the results must be accepted with a great deal of caution.

The idea that the commerce of the Port of New York is declining seems to have originated in the falling off in the export grain trade, and also because the years following the "boom" years, 1879–1883, showed a decline of both values and tonnage. Times of industrial expansion and industrial depression follow each other with more or less regularity, and he who uses statistics should not lose sight of the fact that figures for "boom" years cannot safely be compared with those of "panic" years, if it is desired to obtain trustworthy results.

Nobody questioned that the United States as a whole was growing from 1881 to 1891, yet the total exports of the United States were valued at $18 000 000 less in 1891 than in 1881, and in no intervening year had the figures for 1881 been exceeded.*

The facts are, that, for the four years, 1880–1883, the average value of the foreign commerce of New York was $847 000 000 per year. For the four years 1897–1900 the average value was $1 065 000 000, showing a growth of about 20% in 17 years. The average net tonnage for the four years, 1880–1883, of the vessels engaged in foreign commerce cleared at the Port of New York was 7 085 000 tons, while the average tonnage for the four years, 1897–1900, was 7 390 000 tons. It must be remembered that this is not conclusive evidence that the tonnage of commerce has increased, for part of the increase is probably due to the present use of very large steamers, the net tonnage of which is

* "Statistical Abstract of the United States." 1900.

large compared with their actual tonnage. But, when taken in connection with the increase in values, it is fairly conclusive evidence that the foreign commerce of the port has not declined. Mr. Conrow.

The falling off in the commerce of New York from the "boom" years, 1880–1883, until the "panic" years, 1893–1896, and, during the same period, the decline of commerce on the canals of New York State, gave rise to the belief that the decline (or more correctly speaking, the slow growth) of the commerce of the Port of New York was caused by the decline of commerce on the Erie Canal. In reality, the decline of the Erie Canal has had but little to do with the retarded growth of New York's foreign commerce. The real causes lie deeper, and can be traced in every case to the same sources, namely, the great increase in population living on or near New York Harbor, and the physical limitations of that port.

The ways in which an increasing number of people affect the foreign commerce to its detriment are many. The constantly increasing ferry traffic and the using of slips for local transportation purposes; the use of land on the water-front for bridge abutments; the larger number of piers and bulkheads for the handling of coal, ice, building materials and food supplies necessary for an increasing number of people; the constantly increasing demand for excursion boats with the attendant use of piers; the growing demands of the coastwise trade and of passenger lines of the type of the Fall River and Albany boats; the location of power-houses, gas-works and factories on the water-front; the increasing amount of water-front set aside by law for the use of the Department of Street Cleaning, the Board of Health and the Fire Department, together with sites for floating baths and recreation piers for the city—in addition to that reserved by the United States Government; the increase of wealth and the number of pleasure yachts, using considerable areas for storage; the increasing amount of water-front used by storage warehouses; the growth of shipyards and manufactories of boats, dry docks and other purposes incident to the growth of shipping; the demand for parks and the using of water-front for park purposes;—these are some of the uses which occupy a constantly increasing amount of the water-front.

The aggregate of all these increasing demands is very great, and while the wharfage of the harbor is constantly being increased by the extension of piers, the rebuilding of old piers on better locations and the building of some new piers and bulkheads, it is doubtful if the wharfage of New York Harbor can be increased much faster than will be needed for the domestic commerce of the increasing population.

In 1897 New York had a wharfage of 120 miles, which was three times as great as that of any other port in the world, the second port in size being Liverpool with its 40 miles of wharfage.

Mr. Conrow. As a result of the large amount of water-front required to supply the domestic needs of a vast population, the extent of water-front suitable for foreign commerce is limited. The report of the Board of Consulting Engineers for the Department of Docks, 1897, contains the following:

"In other words, of the 2½ miles of water-front which are better adapted than any other to the use of the class of commerce on which the prosperity of New York is most dependent, only 13.24% is now occupied by foreign steamship lines, and only 27.08% is occupied by all classes of ocean steamships, while 28.89% is occupied by railroads."

As an indirect effect of the increase of population, must be noted the fact that the rate of wages prevailing in New York is higher than in any other port on the Atlantic Coast. The average daily wages of common laborers in New York* for the 5 years ending in 1898 was $1.56 per day; in Philadelphia, $1.50; in Boston, $1.41, and in Baltimore and New Orleans, $1.25 per day; or the average for New York is over 24% higher than in Baltimore and New Orleans. The contrast was even greater a generation ago, and for the 5 years ending in 1874 the average wages in New York for laborers was $1.78 per day; in Boston, $1.32; in Philadelphia, $1.34; in New Orleans, $1.33; and in Baltimore, $1.22. That high wages in New York in the seventies did not have more effect on the commerce at that time was largely due to the crude facilities offering at competing ports.

Laborers are not the only class which receives higher wages in New York than in other ports of the Atlantic Coast. The same authority gives the average wages of teamsters for the 5 years ending in 1898 in New York as $2.10; in Boston, $2.04; in New Orleans, $1.83; in Philadelphia, $1.70, and in Baltimore, $1.56.

An examination of the statistics for all the different trades shows the same fact. Bricklayers in New York receive $4 per day; in Baltimore, $3 per day. Carpenters in New York receive $3.50 per day; in Baltimore, $2.42, and so on through the list.

The Department of Labor states that no statistics of a comparative nature regarding 'longshoremen have been published since 1893. The Seventh Special Report of the United States Department of Labor, 1893, gives the average pay of 'longshoremen in Maryland as $1.37 per day, and the week 65 hours in length, while in New York the average pay is given as $1.78 per day and the week 60 hours long; or comparing them upon the same basis of wages per hour, the rate in New York is 40% higher than in Maryland.

There can be no doubt that wages is an important factor in the shipment of freight, and that the higher wages in New York compared with other ports of the Atlantic Coast—especially those to the

* Bulletin No. 18, Department of Labor, 1898.

southward—has been an important reason for the diversion of commerce to those ports. Mr. Conrow.

The difference in railroad rates of 1 cent per bushel in favor of Philadelphia and Baltimore and against New York, mentioned by Mr. Blackmar, does not seem to need any other explanation of its cause than the greater actual cost to the railroads of delivering grain at New York. In his report, Mayor Symons states*:

"In the transportation of grain from the western grain-producing and storing centers the trunk lines terminating in New York City for many years have made it part of the transportation agreement that they (the railroads) shall deliver the grain alongside of the ship without charge therefor, except as included in the freight rate. In pursuance of this agreement the railroads transfer grain from their cars and through elevators into lighters, by which it is transported in the harbor alongside of vessels and for which transportation or lighterage a charge of 3 cents per 100 lbs., equal to 1.8 cents per bushel of wheat, is exacted by the lighterage companies doing the work."

The same authority states†:

"A statement that the cost of transferring 100 000 bushels of grain from cars to ships at Philadelphia, Baltimore and Newport News is about $275, or a little more that ¼ of a cent per bushel, has been made."

In other words, it costs at least 1 cent per bushel less to transfer wheat from cars to ships in Philadelphia, Baltimore and Newport News than it does to transfer from cars to alongside ships in New York. The fact that the railroads "deliver the grain alongside of the ship without charge" should mislead no one. If the actual cost is 1 cent per bushel greater in New York than in other ports, somebody must pay for it, and it may be taken for granted that this somebody will not be the railroad company.

The differential, then, is not a discrimination against New York, but is merely the recognition of the fact that it costs more to do the same work in New York than it does in other ports of the Atlantic Coast.

Again, the average rental price of 20 piers on the North River‡ is $49 200 per pier per annum. Capitalized at 4%, this represents $1 230 000, as the value of the pier, land under water and wharfage rights. The cost of building such a pier, including the shed, may be roughly taken as $230 000. In other words, the steamship company pays at the rate of 4% on $1 000 000, or $40 000 yearly for the use of the land under water and the wharfage rights of each pier. That this rental price is a serious handicap to the foreign commerce which must compete with ports where the same charges are almost nothing, no one can doubt. Its effect may be better understood when it is stated that if the steamship companies made a clear profit at New York of 1 cent

* Report of State Engineer and Surveyor, New York, 1897.

† *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 305.

‡ Annual Report, Department of Docks and Ferries, New York, 1899.

Mr. Conrow. per bushel on every bushel of wheat exported from New York in 1900, the total profit would have paid the rental of only 4½ piers.

Neither are these piers unusually high-priced, nor will the rentals decrease in the near future. Of the twenty which have been taken, the rental varies from $28 700 to $84 000 per year, and the average term of lease is given as 10 years, with a privilege of renewal for 10 years more. In four cases there is a privilege of renewal for two terms of 10 years each at an increase in price of 5% over the price of the preceding term, and in six cases of the twenty the rental for the second term of 10 years will be 10% higher than the price for the first term.

Even at the high rentals which now prevail, the demand for such piers far exceeds the supply. It will thus be seen that while the port charges of New York are high, the tendency toward still higher levels is very evident. It has often been argued that the port charges of New York should be reduced, but no one can deny that the rental values of piers bear a close relationship to other values in New York—are, in fact, strands of the same net—and he who would lower the port charges of New York would better begin operations by reducing the price of real estate on Broadway.

It is generally recognized that the effect of high port charges is very much greater on articles of large bulk and small value than upon articles of small bulk and great value, and results in the driving away of freight of large bulk. The capacity of the port, then, is further limited by the time required for the handling of the enormous quantities of boxes, bales and bundles of manufactured articles along the North and East Rivers, the great mass of which constitute an increasing proportion of the commerce of the port. These articles are unloaded largely by hand, piled up on the piers by hand, loaded on trucks by hand and carted to the storehouses in practically the same way that prevailed one hundred years ago. As a consequence, the amount of freight which can be handled on each pier is limited.

These conditions are found in the most congested localities on the North and East Rivers, and are caused equally by the export and import trades. In other parts of the harbor, in fact wherever possible, machinery is used for handling freight.

This necessity for movement of freight by hand has greater importance as freight rates are lowered, for it becomes a larger percentage of the cost of transportation.

As an example of the cost of handling goods in New York City, the writer knew of a case recently where an organ was shipped by rail and water from near Springfield, Mass., to New York. The charge for freight for the whole distance was 65 cents. It cost the owner $1.65 to have the organ carted from the pier to his home in the city.

"Under existing modes of distribution, it costs more to deliver a barrel of flour from a railway depot in Jersey City to the home of the

consumer, in Manhattan or Brooklyn, than to convey it from Duluth to Buffalo, a distance of 1 000 miles."* Mr. Conrow.

On the other hand, New York has its compensations, and these have been sufficiently powerful to counteract to a great extent the advantages of competing ports. The greatest advantage consists of the market for imports in New York City and vicinity. Its commercial population of 4 000 000 is four times as large as that of any other seaport in the United States, and furnishes consumers for great quantities of imports. This vast population is likewise the greatest contributor of passengers of any locality in the country. Nearly all lines of steamships carry passengers, and it is of direct advantage to this part of their service to use the Port of New York. Another advantage is found in the ocean rates from New York to Europe, which are cheaper than those from competing ports. Still another advantage lies in the fact that New York Harbor is safer than any other; not only because of greater depth of water, but also for climatic reasons, such as freedom from violent storms, cyclones, etc.—the Galveston storm being an instance of the conditions with which some other ports have to contend.

Another compensation of the Port of New York was brought out in the evidence before the New York Commerce Commission. From New York, a merchant can ship to any port of the world, while rival ports offer connection with only certain foreign ports.

The amount of grain shipped from New York from year to year depends largely upon its destination. If England wants the grain, New York must compete with practically all the ports, while if the principal demand comes from the Mediterranean ports the competition of her rivals is of very much less importance.

One of the main advantages claimed for a ship canal from the Lakes to New York is the abolition of transfer charges at New York. Other advocates claim that an enlarged canal is necessary to maintain New York's foreign commerce. These arguments are contradictory. To New York City the principal advantage of increased exports lies in her port charges. If these charges are abolished, wherein does she benefit from commerce passing through the port *en route* for foreign countries? The "stream of commerce" is a pleasing metaphor, but if it "enriches the country through which it passes," it must do so at the expense of that commerce. The western producer has no intention that his wheat for Europe shall enrich the ports through which it is shipped except to the least possible extent. It is precisely for this reason that he selects cheaper ports than New York.

Every railroad is a "stream of commerce," but of what benefit is it, for example, to the citizens of Syracuse to have long trains of through-freight pass along one of their principal streets? Apparently, it is not a great benefit, judging from the efforts being made to have this "stream of commerce" run in a less conspicuous channel.

*"New York Commerce Commission," 1900.

Mr. Conrow. Nor would a ship canal benefit New York. Such an amount of tonnage as the estimates contemplate passing up and down the Hudson would further aggravate the already congested conditions of the port—and this, in addition to the withdrawal to interior harbors of trade which New York now enjoys, would constitute a serious menace to the foreign commerce of New York.

Another phase of the canal question, which it is of great importance to consider, is the fact that the export grain of the United States represents largely the cream of fertility of the West, and when the skimming process has been completed the production of grain will decrease. The principal difference between the methods of the eastern and western farmers is that in the East the farms are fertilized, and in the West (at least in the great wheat-growing areas) they are not. The wheat grown in the Dakotas, for example, is the product of the natural fertility of the soil. The natural fertility will not last forever, or, indeed, for any considerable term of years, as 10 years of constant raising of the same crop will exhaust the richest soil if no fertilizer is used. The United States exported 217 000 000 bushels of wheat in 1898, and the three States, North and South Dakota and Minnesota, produced 176 000 000 bushels, or more than four-fifths of this amount.*

The writer has known of many cases of exhaustion of the soil. Formerly, great quantities of peaches were raised in Southern New Jersey, while now very few are, because they can no longer be successfully grown. A single orchard of peaches will exhaust the soil for several generations for that particular crop. The writer knew a man who owned a large tract of timber of 50 or 60 years' growth. Thinking to obtain a plot of "new" ground he cut off the timber and planted peach trees on the area. The peach trees never thrived at all, and after trying in vain for several years, the orchard was abandoned. Happening one day to look up some old records, the farmer found, much to his surprise, that a peach orchard had occupied the same ground 70 or 80 years before. The writer knows of fields where formerly the egg-plant yielded large crops and where now that vegetable cannot be matured at all.

This exhaustion of the soil is not peculiar to peaches and egg-plants, but applies in different degrees to every food product. The wise farmer in the East "rotates" his crops. He does not raise wheat after wheat, but lays out his farm mathematically, and plans that a certain crop will come in a certain field only once in 6 or 7 years. By this method, and by continually enriching, the fertility of the soil may be maintained for centuries, as has been the case in Europe.

The day of reckoning for the western farmers is near at hand, when they also must put back on the soil an equivalent of that which they take from it; and when that time comes the price of wheat must increase

* "The Grain Trade of the United States," 1900.

or production will fall off. In either case the amount exported will be less than at present. Mr. Conrow.

The United States has never yet had to face this question as it must within the next generation. Up to this time the answer has always been the same—"go further west." But, beyond the Dakotas, in the United States, the wheat area cannot go, because of the Rocky Mountains.

In 1840 the center of the wheat-growing area was just east of Pittsburg. In 1898 the five States, North and South Dakota, Minnesota, Nebraska and Kansas, produced over 40% of the total wheat crop of the United States. In 60 years the center of production has moved westward about 1 000 miles. Through New York, Ohio, Indiana, Illinois, Iowa and the Dakotas the wheat center has successively passed, and each region has in its turn contributed of its best fertility.

The western movement of the wheat-producing area has been accompanied by the decline of the amount raised in the East. In 1840 New England produced 2 800 000 bushels of wheat, and in 1898 only 140 000 bushels. In 1849 New York State produced 13 000 000 bushels, and in 1899 only 7 000 000 bushels. The reaction following close behind the great western movement can be traced as far west as Illinois, which reached its maximum production in 1879, when it raised 51 000 000 bushels of wheat. In 1899 its production had fallen to less than 13 000 000 bushels. The reason for this decline in the East is that the farmers there cannot afford to raise wheat at the prevailing prices. A successful farmer of long experience has furnished the data upon which are based the following figures relating to the cost of producing 1 bushel of wheat in the Eastern States:

Interest and taxes on land, at $100 per acre	$5.00
Cost of fertilizers, per acre	4.00
Labor, per acre	4.00
Housing, per acre	2.00
Total cost, per acre	$15.00
Cost per bushel	60 cents.

The yield* may be taken as 25 bushels per acre. At 62 cents per bushel (the average price of wheat for the 5 years 1894–1898), the value of the crop, per acre, amounts to $15.50, showing a net profit of 50 cents per acre. It is obvious that, at this price, there is no inducement to raise wheat in the East. Only under the most favorable conditions of enormous acreage, fertile soil and wholesale methods can wheat be produced profitably at present values. The only reason that many farmers in the East continue to raise wheat at all is to obtain the straw, which is a necessity for their farm stock.

Wheat will not support a dense population. The average consump-

* The farmer who furnished these data stated that his average yield for forty years was greater than this, though this figure is nearly double that given by the Government statistics as the average yield per acre in the Eastern States.

Mr. Conrow. tion of wheat and wheat flour in the United States, extending over a long term of years, has been about 5 bushels for each inhabitant per year. The average yield per acre in 1898 was 15.2 bushels, or, an acre of wheat would supply the wants of three persons.

On the other hand, the average yield of potatoes in New York in 1900 was 81 bushels per acre, which quantity would support a very much larger number of people per acre. In New York, in 1899, an average acre of wheat was worth $14.83, while an average acre of potatoes in 1900 was worth $36.45.

In 1839 New York was the third State in the production of wheat; in 1899 it ranked twenty-third. In 1900 the production of potatoes in New York was greater than in any other State. This tendency to raise more concentrated foods will increase as the population becomes more dense.

The falling off of the exports of wheat and other grain from New York has been in a measure due to geographical reasons. Kansas is as near the Gulf as it is to Chicago. Why should a shipper in Kansas send export grain through Chicago—no matter how low the charges are from there to New York—when he could reach the Gulf for the same cost as he could reach Chicago? The Chicago–New York rate would have to be almost nothing to make the two routes equal. The same is largely true of Nebraska, while Oklahoma is nearer the Gulf than Chicago. Yet these three political divisions produced, in 1899, 73 000 000 bushels of wheat. Allowing a domestic consumption of 5 bushels per inhabitant, they had a surplus of about 60 000 000 bushels, or more than 27% of the quantity exported from the United States in 1899.

In the United States the export trade in wheat is not growing rapidly. The year of greatest exports was 1892, when over 225 000 000 bushels were shipped out of the country. Since that time this quantity has not been exceeded, in spite of the fact that the production in 1899 was 64 000 000 bushels greater than in 1892.

Assuming that the production of wheat of the United States can be maintained at its present rate, using 1 200 000 as the yearly increase in population and taking the consumption at 5 bushels annually *per capita*, by 1930 the United States will produce only enough wheat for its own consumption.

These are matters which bear a close relationship to the future development of the commerce of the United States.

Mr. North states:*

"Hence if we can cut four or more cents off the cost of marketing our wheat, while our rivals cannot decrease the cost of marketing theirs, our producers, and not foreign consumers, will have the sole advantage"; and again, "Any reduction in the cost of transportation, due to an enlargement of the Erie Canal for export purposes would

* *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 290.

influence the entire crop and be divided between producer and consumer." Mr. Conrow.

Why is it, then, that the price of wheat has steadily declined since 1870 or before, in spite of the fact that transportation rates are now but a fraction of what they were then? For the five years, 1866–1870, the average price of wheat in the United States was $1.15 per bushel, and for the five years, 1894–1898, the average price was 62 cents per bushel. During the same period, the rates on wheat from Buffalo to New York decreased from 11.2 cents per bushel in 1870 to 3 cents per bushel in 1899.* In what way has the producer benefited by the decline in transportation charges?

Bearing upon this question, the writer has known of the following: About ten years ago some fruit-growers near Philadelphia discovered that the Boston market was often much higher than the Philadelphia market for early summer apples, and that the cost of transportation from Philadelphia to Boston by a newly-established line of steamers was 25 cents per barrel. The news quickly spread, however, and so many apples were sent to Boston that the price there became actually lower than in Philadelphia. Conditions soon became normal, however, and the price of early summer apples in Boston is now 25 cents per barrel—plus the profits and risk of shipment—higher than in Philadelphia. At the time these cheap transportation rates were inaugurated it was prophesied that the producer would be greatly benefited by receiving higher prices, but such has not been the case, for there has been a steady decline in the price of this fruit. Evidently, the major part of the benefit has gone to the consumer. The producer has received no direct benefit through advanced prices, but he has received benefit from a broader market.

Formerly, it frequently happened that summer apples and perishable vegetables of all kinds were so plentiful in Philadelphia that great quantities were wasted. This happens very seldom now, for cheaper rates in all directions have enormously increased the market for such products; but the prices have steadily declined. It was not unusual fifteen years ago for tomatoes to sell for $4 per bushel in Philadelphia, in June, but cheap and fast transportation has transformed these conditions until now such prices are seldom obtained in midwinter. Formerly, the producers of southern New Jersey, Delaware and Maryland received large profits whenever they succeeded in producing early crops. Now, the farmers of Georgia and Florida sell tomatoes in New York in January. Prices, generally, for perishable food products have steadily declined, along with the decline in transportation rates. But while the farmer of fifteen years ago could sell only a few hundred bushels without breaking the price, he can now dispose of several thousand bushels—at a lower price it is true—but without affecting the market.

* "The Grain Trade of the United States," 1900.

Mr. Conrow.

On every hand the evidence points to the same conclusion: That the direct benefits of cheaper transportation are received by the consumer in lower prices, while the producer receives no other benefit than that of a broader market.

Mr. Mayer says*:

"The Northwestern States are vitally interested in reducing freight rates to Europe, * * * since the home price of produce which is exported in considerable quantities is equal to the price in the principal foreign markets, less the cost of reaching them."

This statement can only be accepted as true in part, for it assumes that the price is fixed entirely in Europe, which is not the case.

The price of a staple article, such as wheat, is the resultant of many forces, of which no doubt the greatest for wheat is the demand of Europe. But the demand of the United States is also an important factor in the equation, and it would be much nearer the truth to say that, of the conditions which fix the price of a commodity, each country contributes in proportion to its consumption of that commodity.

Europe, including the Russian Empire, consumes about four-fifths of the total wheat production of the world, and the United States about one-seventh.† Therefore, of the 35 elements which determine the price of wheat throughout the world, Europe contributes 28, the United States 5, while the remaining 2 come from various other countries.

As the United States increases in population its influence in fixing the price of wheat will be greater and its dependence upon European markets less.

There are two important factors to which the advocates of deeper canals have not given sufficient attention. The first is the enormous disadvantage of all canals, when compared with railroads, in the matter of speed. It is a disadvantage which cannot be eliminated. Time is money, and contemporaneous with industrial growth in every direction has been the increase in the value of time.

The writer was surprised when in Brockton, Mass. (christened the Shoe City), to learn that the manufacturers there should consider it a matter of vital importance that the railroad should maintain a special fast freight leaving Brockton every afternoon at about 4.30 o'clock and connecting with the Fall River boat for New York. In this way, shoes made one day in Brockton were in New York by the next morning. At first it does not seem that this great haste is necessary, but when it is remembered that the majority of shoes conform to certain styles and that a style is in vogue for only two or three months, it is seen that every day is an appreciable percentage of a season.

If the cost for wheat by all-rail from Chicago to New York is 10 cents per bushel and the time 3 days, and the cost by Lake-canal 5 cents per bushel and the time 8 days, the difference in favor of the

* *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 273.

† "The Grain Trade of the United States," 1900.

Lake-canal route is not 5 cents per bushel, but 5 cents less the value of 5 days, during which time the wheat is not available for delivery. It has sometimes happened that the market price has fluctuated more than 5 cents per bushel in 5 days. Mr. Conrow.

The second element which has received scant attention from some advocates is the question of management. All are agreed that no canal would succeed under the same conditions that have retarded the growth of commerce on the Erie Canal. What guarantee is there that conditions in the future will be any better than those of the past, if the State of New York constructs the deeper canal.

Business does not thrive upon uncertainties. When asked by the New York Commerce Commission, what was the objection to using the Erie Canal, a grain merchant testified: "The canal has been a hard thing to depend on; you can never tell when the grain would get here."

The figures of Mr. Rafter* show that over a period of 10 years the average delays due to breaks have amounted to 25 days for each canal season—or for every 8½ days the delays caused by breaks have amounted to 1 day. It is hardly cause for wonder that commerce has declined under such conditions.

The Erie Canal has never had a fair trial. No one knows how cheaply it could transport freight, if properly developed.

Mr. Foote,† after many years' experience in the transportation business on the Erie Canal, pointed out that the present capacity of that canal, as determined by New York State officials, is 18 000 000 tons per annum, and this could be doubled without deepening the canal. It was stated before the New York Commerce Commission that a canal 9 ft. deep would have a capacity equal to that of five trunk lines of railways. This is so far in excess of any probable traffic that its present capacity is ample for many years. Mr. Foote also states that by using the "steam train-towage system," by submerged wire cables, a speed of 8 to 10 miles per hour may be obtained; a speed which compares favorably with that estimated for the deeper canals.

Major Symons states:‡

"That the traffic on the Erie Canal has declined, it is true, but it would immediately spring up and grow to enormous dimensions if a proper barge canal were built in its place, if legislative restrictions on the capital of operating transportation companies were removed, and if the business were organized in a thoroughly up-to-date manner." And again: "There are decided reasons for believing that it would be more convenient, and better for commerce, to have nearly all the western products arrive in New York in lots of about 1 000 tons, in place of having them arrive in lots of 6 000 to 10 000 tons."

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* *Transactions*, Am. Soc. C. E., Vol. xlv, p. 298.

† *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 297.

‡ *Transactions*, Am. Soc. C. E., Vol. xlv, pp. 240 and 242.

Mr. Conrow. These remarks apply as well to the present canal as to a barge canal; for the fact remains that the great mass of freight transferred in New York Harbor is done upon craft which draw less than 7 ft. of water, and the present canal-boat is well adapted for such conditions.

The consideration of these many phases of the question has led to the following conclusions:

I. That the foreign commerce of the Port of New York is slowly growing.

II. That, of the many causes which retard the growth of foreign commerce at New York, the greatest are found in the increasing population living on New York Harbor, and in the physical limitations of the port.

III. That a ship canal connecting the Lakes and New York City would be detrimental to the foreign commerce of the Port of New York.

IV. That a barge canal would mainly benefit consumers in New York and the adjacent States; that it would have but slight influence on the foreign commerce of the Port of New York; that it should be undertaken only by the Federal Government.

V. That the Erie Canal has never had a fair trial; that, with its present depth, it is capable of handling economically a large tonnage; that the State of New York should abandon all schemes for canal enlargement; should guarantee some definite policy for the future and should foster and encourage the development of commerce on its present canals.

Mr. Rafter. GEORGE W. RAFTER, M. Am. Soc. C. E. (by letter).—The Report of the New York Commerce Commission, transmitted to the Legislature, January 25th, 1900, is in reality a discussion, filling two octavo volumes of more than 2 000 pages, of the questions considered in this paper. In his message transmitting the report of this Commission to the Legislature, Governor Roosevelt said:

"This report represents an immense amount of wholly disinterested labor, undertaken solely with a view of accomplishing something effectual to stop the decline in the commerce of New York. The thanks of the State are due the members of the Commission for the marked ability and untiring industry they have shown throughout their labors."

The report of this Commission is included in 141 pages, the remainder being various appendices of the testimony taken, etc. Of these 141 pages, nearly one-half is occupied with a discussion of the effect of the differential, the remainder being mostly taken up with an account of various drawbacks which exist at New York, and which can only be remedied by the action of the people of New York themselves; for instance, port charges, among which are included "the one-cent arbitrary," "elevator charges," "lighterage charges," etc.

Mr. Rafter.

The following presents a summary of the conclusions of this Commission:

(1) The decline in New York's commerce has been steady and continuous for many years; it has been more pronounced during recent years, and has now reached serious proportions in an actual loss of exports. This loss has been largely in exports of grain and flour. While New York has been steadily losing, Montreal, Boston, Baltimore, Newport News and the Gulf ports of New Orleans and Galveston, have made substantial gains.

(2) The loss to New York is due in great measure to a discrimination against New York in railroad rates imposed by an agreement, known as the differential agreement, between the trunk line railroads of the American Atlantic seaports including the New York railroads.

(3) While this differential agreement, by its terms, gives preference in railroad rates only to Philadelphia, Baltimore, Norfolk and Newport News, the Port of Montreal and the Gulf ports are benefited, to the injury of New York, to as full an extent as if they were parties to the agreement, in view of the fact that in the competition with the Gulf ports and with Montreal the differential agreement alone prevents the New York roads from giving New York as low a rate as is accorded by the agreement to Baltimore and to Newport News; and the same differential agreement, while providing the same rate for Boston as for New York, permits the Boston roads to allow free insurance and free storage to an extent that amounts to as substantial a discrimination against New York as is imposed by the discrimination in the rail rate in favor of Philadelphia, Baltimore and Newport News.

(4) As a result of this situation, in which New York is prevented from receiving the benefit of her natural advantages, in competition with these other ports, each of the ports named has made gains, while New York has continued to lose.

(5) While there are port charges in New York that can and should be reduced, it will be impracticable to assure to New York any benefit from such reductions so long as the railroads are permitted to offset the saving in expense, thus secured, by correspondingly increasing the differential against New York. That such is the principle of the differential agreement and the policy of the New York roads, as well as the roads to the Southern Atlantic ports, was conceded by the officials of the New York roads when they appeared before the Commission.

(6) The theory of such discrimination is, that, under natural competitive conditions, New York would maintain its pre-eminent position in the export as well as in the import trade of the country—New York under such natural conditions being the cheapest route to the foreign markets for the products of the West. To the extent that this differential agreement requires the New York roads to charge a higher rate than otherwise would be imposed upon the export products of the West, it is in effect an export tax upon such products.

(7) To abolish the differential, therefore, would not only result in New York regaining the commerce now diverted to other ports, but it would also benefit the producers and exporters of the West when competing in the foreign markets. This is explanatory of the fact that the Western exporters express themselves as equally con-

Mr. Rafter. cerned with New York regarding this railroad discrimination against New York.

(8) The principle of the differential agreement is inequitable and unjust, in theory and in practice. New York has suffered much therefrom, and should use every means within her power, not only to have it abolished, but also to render it impracticable of restoration. The differential rate applies not only on products destined for export, but also destined for local consumption by the people of New York City.

(9) The fact that the New York Central and Hudson River Railroad Company has received exceptional benefits from the State of New York makes that corporation exceptionally culpable for participating in the discrimination to the serious injury of New York.

(10) This railroad discrimination against New York would be impossible without the participation of the New York Central and Hudson River Railroad Company, and the demand that it withdraw irrevocably from the differential agreement is now made understandingly.

(11) The State has it within its power, through an adequate improvement of the Erie Canal, not only to apply the remedy that will secure it against further loss of its commerce, but that will secure to it as well the restoration of that which has already been diverted.

(12) This result may be achieved through the completion of the improvement of the State canals contemplated by Chapter 79 of the Laws of 1895, at an expenditure not exceeding $15 000 000. To receive full benefit from the completion of such improvement, canal terminals should be provided by the State, both at Buffalo and at New York, for the receipt, safe-guarding and delivery of package freight transported or destined for transportation over the Erie Canal.

(13) While thus providing for competition with rail rates, sufficient to render difficult if not impossible a discrimination against New York, certain high charges at the Port of New York should be reduced, including the charge for elevating grain, and also including the excessive rentals imposed for the use of public docks.

* * * * * * * * *

These thirteen propositions could be much more widely discussed than they have yet been. While the great industry displayed by this Commission is willingly conceded, it is the opinion of the writer that some of these propositions are hardly true.

Mr. Blackmar. ABEL E. BLACKMAR, Esq. (by letter).—An extended reply to the criticisms on the paper, and to the discussions, does not seem necessary. Such a reply would be, to a great extent, a repetition and enlargement of the matter contained in the paper itself. A short summary, however, may not be out of place.

It is not disputed that the system of differentials was established as stated in the paper, nor that it was intended to be discriminative against the Port of New York, which it necessarily is, in its very nature, nor that it has had its intended effect. As traffic flows along the line of least resistance, it needs no elaborate demonstration to

show that the resistance of an arbitrary differential in freight rates tends to direct traffic to other routes; and if such traffic would have flowed to the Port of New York, except for the resistance of the differential, it may with propriety be said to be "tributary" to New York and to be "diverted" to other points. Mr. Blackmar.

To one who believes that it is for the railroads to determine arbitrarily the route of commerce, to take it from the merchants of one locality and give it to those of another, or to one who is indifferent to the relative prosperity of New York and the other Atlantic seaports, the differential may not seem an evil.

The writer has not accused the railroad companies of moral wrong or of hostility to the Port of New York in imposing the differentials. They have undoubtedly done the best possible from the point of view of their own interests; but if, in seeking their own interests, they have adopted a policy injurious to the merchants of New York, this is certainly an evil needing a remedy, from the point of view of the merchants.

The paper does not purport to discuss generally the cause of the decline of commerce in the Port of New York. Many causes have doubtless contributed to produce the decline and many causes have resisted it. It is impossible to analyze the situation completely and to assign to each factor its true value in the problem. In the case of the New York Produce Exchange against the Railroad Companies, referred to several times in this discussion, the testimony was to the effect that ⅛ cent a bushel difference in the costs of through transportation is enough to determine through which seaport export grains will move. The differentials have always been more than this. They are, therefore, a sufficient explanation for the decline in the exports of wheat, corn and flour from New York, which the statistics show. All other causes are apparently more than offset by the natural and acquired advantages of the port. As the decline in commerce is admitted and the differentials are sufficient to produce it, the writer sees no reason for painfully seeking other causes in excuse of the differentials.

It is stated in one of the discussions that for a number of years there have been no differentials on ex-lake grain for export. The information of the writer is to the contrary of this statement. It is unfortunate to differ on a question of fact. The writer relied on information furnished to him, at the time the paper was written, by the Interstate Commerce Commission and by merchants engaged in the business of bringing the grain from the West to the seaboard for export.

On September 16th, 1901, the writer made inquiry of the Interstate Commerce Commission, and received the following:

"*All-Rail Grain Rates, Chicago and Buffalo to New York, Philadelphia and Baltimore.*—Referring to the accompanying letter of Mr. Abel E. Blackmar, * * * examination of tariffs on file with the Com-

Mr. Blackmar. mission shows the all-rail rates on grain, from and to points mentioned, to be as follows:

	In cents per 100 lbs. Domestic.	Export.
"From Chicago, Ill.:		
To New York	15	13½
Philadelphia	13	12½
Baltimore	12	12
"From Buffalo, N. Y.	In cents per 100 lbs.	
To New York	9	
Philadelphia	8½	
Baltimore	8½	

"Respectfully submitted,

"(Signed) J. M. SMITH,

"*Auditor.*"

Inquiries also of merchants show that the ex-lake differential, at all times since its establishment, has been maintained upon export grain from the Lake ports to New York, Philadelphia and Baltimore.

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SHORT AND EASY METHODS FOR COMPUTING PROBABLE ERRORS.

By E. A. Fuertes, M. Am. Soc. C. E.

The engineer is constantly engaged in a large number of professional problems, the solutions of which depend upon the averages of experiments based upon a certain number of repeated observations, as, for example, when gauging the delivery of a stream by taking many velocity measurements; making a series of tests on cement briquettes; or experimenting with the strength of materials, etc.

Usually, the means of various results are classified, in some more or less arbitrary manner, and these means are accepted as the best obtainable value; but the engineer is not able to determine the actual degree of confidence to be placed in the average value thus obtained; neither can he determine whether the relation between the individual results and their means are the proper ones, nor that the number of observations and experiments are sufficient to justify the blind adoption of their means.

Generally, it has been deemed necessary to apply the method of least squares to discuss the results of important experiments. This method is generally long and tedious, and many engineers are not familiar with it, even if they ever learned it at school. Also, though

the advantage of the rigorous method of least squares is easily granted, often its application would be entirely useless, because the labor and time required would seem unjustifiable in a very large number of cases in engineering investigations; and yet it would be always desirable to learn if the various observations are consistent with each other, and if their mean is entitled to the proper degree of confidence.

If this useful information could be obtained with ease and rapidity, there is no doubt that the engineer would avail himself of its advantages; and the absurd and even dangerous deductions, sometimes based upon insecure averages, would be less numerous, thus fostering reliable progress in a large number of engineering investigations.

The origin of the method herein explained has an interesting history, connected with our Civil War in an indirect way.

When Napoleon III conceived the Quixotic notion of re-establishing the supremacy of the Latin races over our Western Hemisphere he took advantage of the disordered condition of the United States, during our Civil War, to enlist against us the hatred and fear of Spain with regard to her then West Indies possessions, and spurred the commercial jealousies of England, to form a triple alliance between France, England and Spain, aiming at the destruction of our national supremacy.

During the darkest days of the Civil War, and when we were unable to prevent it, the allied armies landed in Mexico, to found a new monarchy upon our southern frontier.

For reasons rather foreign to the objects of this paper, Spain and England withdrew their forces from American soil, leaving Napoleon in the lurch. The Emperor Maximilian had not yet lost the prestige of that formidable alliance, when the Mexican armies had taken to the woods, initiating the guerilla warfare which culminated in the capture of the Emperor, the collapse of the Empire, the pathetic madness of the Empress Carlotta, and the death of the misguided and betrayed Emperor. The Mexican armies operated in the fastness of virgin, tropical forests, and their movements could be guided only by the astronomical work of F. D. Covarrubias, a Mexican engineer of great ability and still greater originality.

The sextant used by him for the purposes of directing the peregrinations of the Mexican forces became injured beyond repair by the fall.

over a precipice, of the mule carrying his field equipment. Mr. Covarrubias, being unable to find the position of his army by any astronomical process involving the knowledge of angular measures, set to work on the spot to develop new methods of computation with entire independence of the graduation of the sextant, using only chronometer times and the almanac data for the solution of astronomical triangles. He felt obliged to modify many astronomical formulas, including those necessary for the discussion of results by least squares. These latter formulas he made exceedingly simple of application, and yet capable of furnishing all the necessary accuracy for a large number of cases in engineering practice.

There are several least-square formulas for various purposes, generally known by the names of their inventors; but they all demand considerable labor on the part of the computer. The Covarrubias formulas can be reduced to a table demanding only a few seconds of time to learn:

First.—The probable error of each individual result in any series of experiments;

Second.—The probable error of the mean of all the results;

Third.—They furnish a fair criterion for the number of repetitions required to reduce the error of the mean to any desirable limit of accuracy;

Fourth.—This method avoids the necessity of squaring each one of the residuals, or the squares of the difference between each result and the mean of all the results;

Fifth.—It avoids the necessity of taking the algebraic sum of the residuals and nearly all the long and tedious processes of least squares;

Sixth.—Any pocket table of logarithms can be used to find the probable errors, without the need of interpolations;

Seventh.—The formulas can be reduced to a table, using as a single argument the number of observations taken.

The formulas chosen by Covarrubias are:

$$r = \pm 0.8453 \frac{[V]}{\sqrt{n(n-1)}} = F[V] \quad \ldots\ldots (1)^*$$

$$r_o = \frac{r}{\sqrt{n}} = f r \quad \ldots\ldots (2)$$

* See Doolittle's Practical Astronomy, p. 21. It may be often desirable to use: $r = \frac{0.85[V]}{n}$.

In these formulas:

r = the probable error of each single result;

r_o = the probable error of the mean of all the observations or results;

V = the residuals, or the numbers obtained by subtracting each result from the mean of all of them (without regard to signs);

$[V]$ = the arithmetical sum of the residuals;

n = the number of observations taken.

The values of F and f, reduced to a table for any values of n, are given in Table No. 1.

TABLE No. 1.—LOGARITHMS OF F AND f FOR COMPUTING PROBABLE ERRORS.

n.	Log. F.	Log. f.	n.	Log. F.	Log. f.
2	9.7765	9.8495	26	8.5206	9.2925
3	9.5379	9.7614	27	8.5038	9.2843
4	9.3874	9.6990	28	8.4877	9.2764
5	9.2765	9.6505	29	8.4722	9.2688
6	9.1884	9.6109	30	8.4572	9.2614
7	9.1154	9.5774	31	8.4428	9.2543
8	9.0529	9.5485	32	8.4288	9.2474
9	8.9983	9.5229	33	8.4152	9.2407
10	8.9499	9.5000	34	8.4020	9.2343
11	8.9063	9.4793	35	8.3892	9.2280
12	8.8667	9.4604	36	8.3768	9.2218
13	8.8304	9.4430	37	8.3648	9.2159
14	8.7970	9.4269	38	8.3530	9.2101
15	8.7659	9.4119	39	8.3416	9.2045
16	8.7369	9.3979	40	8.3304	9.1990
17	8.7097	9.3848	41	8.3196	9.1936
18	8.6841	9.3724	42	8.3090	9.1884
19	8.6600	9.3606	43	8.2987	9.1833
20	8.6371	9.3495	44	8.2886	9.1783
21	8.6154	9.3389	45	8.2787	9.1734
22	8.5947	9.3288	46	8.2690	9.1686
23	8.5749	9.3191	47	8.2596	9.1639
24	8.5560	9.3099	48	8.2503	9.1594
25	8.5379	9.3010	49	8.2413	9.1549
			50	8.2324	9.1505

Example I.—The following five observations were taken to find the error of a clock:

Observation.	Clock fast.	V = residuals.	
	Min. Sec.	Sec.	The residuals V are obtained by finding the difference between each clock error and the mean of all the errors.
I	16 41.72	0.18	
II	16 41.18	0.36	
III	16 41.15	0.39	
IV	16 41.60	0.06	
V	16 42.06	0.52	$[V]$ = the arithmetical sum of the residuals.
	16 41.54 = mean	$[V]$ = 1.51 = sum	

Since, in this example, there were five observations, we take out the values of F and f corresponding to $n = 5$, proceeding as follows:

Log. F..... 9.2765 from the table,
Log. $[V] = (1.51)$ 0.1790 = log. of sum of residuals,
Sum..... 9.4555 = log. r: hence, $r = \pm 0.28$ second,
Log. f..... 9.6505 from the table,
Sum..... 9.1060 = log. r_o: hence, $r_o \pm 0.13$ second.

This shows that the probable error of each single determination, $= r$, is within 0.28 second, more or less; and the probable error of the mean of all observations, $= r_o$, can be depended upon to be true within ± 0.13 second, that is: considering the number and nature of the observations; the accuracy with which the decimals have been carried out in the original time computations; the outstanding errors of the stars chosen, as well as of the instruments used; accidental errors and every other circumstance affecting the error-producing causes attending all classes of observations, the probable errors, r and r_o measure the degree of reliability of the work.*

The value of r must vary with each individual observer in every case in which the result depends upon personal skill; but it is very apt to remain constant for the same observer after he has acquired a reasonable degree of practice in the special manipulation of the work in hand.

If, in the above example, the observer should require a better degree of precision in the probable error of the mean than 0.13 second, since, from the value found for r, he knows that the probable error of each observation is about 0.3 second, if he should now desire to reduce the error of the mean to 0.07 second, he must make

$$r_o = \frac{r}{n} = 0.07 \text{ second},$$

Solving for n, we get $n = 16$; because $r_o = \frac{0.3}{16} = 0.07$ second.

Therefore, the observer must take 16 observations, instead of 5, to obtain the desired result.

Example II.—The following observations were taken to ascertain the relation between the velocity of water, as derived directly from its actual weight when passing through a known cross-section in a given time, and the velocity as deduced from Pitot tube observations, viz.:

$$\frac{\text{Velocity by weight}}{\text{Velocity by Pitot tube}} = \text{Coefficient}.$$

*The rigorous method of least squares gives $r = \pm 0.26$ second, and $r_o = \pm 0.12$ second.

Set A.		Set B.	
Coefficient.	Residuals.	Coefficient.	Residuals.
0.8400	0.0055	0.8280	0.0020
0.8349	0.0004	0.8260	0.0000
0.8409	0.0064	0.8328	0.0068
0.8430	0.0085	0.8299	0.0039
0.8476	0.0031	0.8179	0.0081
0.8359	0.0014	0.8268	0.0008
0.8245	0.0100	0.8346	0.0086
0.8223	0.0122	0.8139	0.0121
0.8348	0.0003	0.8310	0.0050
0.8213	0.0132	0.8190	0.0070
Sum = 8.3452	$[V] = 0.0620$	Sum = 8.2599	$[V] = 0.0543$
Mean = 0.8345		Mean = 0.8260	

Entering the table with $n = 10$, we have, for each set:

Set A.	Set B.
Log. F..... 8.9499 from the table,	Log. F..... 8.9499 from the table,
Log. $[V]$..... 8.9724	Log. $[V]$..... 8.7348
Log. r = sum..... $\overline{7}.9223$, hence, $r = 0.0083$,	Log. r = sum =... $\overline{7}.6847$, hence, $r = 0.0048$,
Log. f..... 9.5000 from the table,	Log. f =..... 9.5000 from the table,
Log. r_o = sum..... $\overline{7}.4223$, hence, $r_o = 0.0026$.	Log. r_o = sum =.. $\overline{7}.1847$, hence, $r_o = 0.0015$.

The foregoing is a very instructive result, for, as may be learned by remarks entered in the notebook in which the observations were kept, the first set, A, was not so good as Set B; but the simple use of the table establishes the fact that the probable error of each single experiment in Set A is about twice as large as those of Set B, while the probable errors of the means in each set differ by a very much smaller amount.

If we now use Sets A and B as a single set, we shall get, using $n = 20$:

Log. F..... 8.6371 by the table,
Log. $[V] = (0.1169)$..... 9.0678

Log. r..... 7.7049, hence $r = 0.0051$,
Log. f..... 9.3455 by the table,

Log. r_o..... 7.0504, hence $r_o = 0.0011$;

from which we learn that the probable error of the single result is smaller than even the average of the previous values for each set; while the probable error of the mean result of the entire sets, $A + B$,

is less than one-half of the value found by the averages of the independent sets. This establishes the usefulness of Set A to be considered as tending to give accuracy to the final result, and thus the best value of the coefficient sought is 0.83025 ± 0.0011.

No claim is made for originality: The intention of the writer has been only to place an easily handled tool in the hands of members of the Society. Extensive practice has proved the usefulness of this approximate method for the discussion of experiments.

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No. 906.

THE LAWRENCE, MASS., CITY FILTER: A HISTORY OF ITS INSTALLATION AND MAINTENANCE.

By Morris Knowles, Assoc. M. Am. Soc. C. E.,
and
Charles Gilman Hyde, C. E.

Presented June 5th, 1901.

WITH DISCUSSION.

Introductory.—The preparation of this paper has been undertaken in order that the story of this interesting engineering work, and, particularly, an account of its operations, might be recorded in one narrative, with all the necessary data at hand for a study of conditions and effects produced thereby.

One of the writers was a member of the Lawrence Water Board* for five years, from January 1st, 1895, and, during that time, had opportunity to learn the details of the cost of operation and the results of changes in methods. The other has had occasion to study the records and determine the efficiency of the filter from the standpoint of the unbiased investigator.

* The members of the Water Board for 1901 are Mr. A. S. Robinson, President, and Messrs. Daniel Gallagher, J. J. Desmond, Edward Arundel and Charles Donovan, to whom the writers are indebted for allowing much of the information in the files of the Board to become available.

Many of the data and statements in portions of the paper preceding and including "Description and Construction" have been largely compiled from more complete descriptions in other publications. This action appears amply justifiable on account of the value of having these data, for which no originality can be claimed, in comprehensive form for reference and study. The writers make this statement in order that there may be no misunderstanding as to the authority for all such information, the credit for which has been given in each case.

Lawrence.

The City of Lawrence is situated in the northeasterly part of the State of Massachusetts, not far from the New Hampshire boundary line. It is the second largest city in Essex County, and ranks as the seventh in population among those of the State. The entire area, which is about 7.3 sq. miles, is rather compactly built up, the larger and denser settlement being on the northerly bank of the Merrimac River, with a smaller and now rapidly developing section south of the stream. The growth of the city since its first settlement in 1843 has been vigorous, and the statistics of the population, as given by the successive census reports, are given in Table No. 1.

TABLE No. 1.—Population of Lawrence, Mass.

Date.	Enumeration by	Population.
1847	Town.	3 577
1848	Town.	5 949
1849	Town.	7 225
1850	U. S.	8 282
1855	Mass.	16 114
1860	U. S.	17 639
1865	Mass.	21 698
1870	U. S.	28 921
1875	Mass.	34 916
1880	U. S.	39 151
1885	Mass.	38 892
1890	U. S.	44 654
1895	Mass.	52 164
1900	U. S.	62 559

The city is distinctively a manufacturing community, and its population, like others of New England having similar industries, embraces, besides native Americans, many of foreign descent. A large proportion of the people has lived in well-constructed tenement buildings, owned by the mill corporations, and maintained, particularly during the early history of the town, under favorable sanitary regulations; all of which

tended to promote a state of general good health, superior to that of many manufacturing cities of this character. At the present time, while there is a restricted portion in which the conditions are more like those which usually obtain in thickly populated factory cities, the foregoing statement in regard to the hygienic conditions is true for most of the city.

Merrimac River.

Water-Shed.—The Merrimac River is formed by the union of the Pemigewasset and Winnipiseogee Rivers, near the central part of New Hampshire. Thence, it takes a generally southerly course to a point in the neighborhood of Lowell near the Massachusetts–New Hampshire boundary line, where it turns sharply and flows in a northeasterly direction to the Atlantic Ocean. The source of the first-named tributary is in the southwesterly slopes of the Franconia Mountains, while the latter flows from a large lake of the same name, and the water-sheds of both rivers are very rugged. As the main river flows southward, the country becomes less mountainous, until in the southerly portion of New Hampshire and the northerly part of Massachusetts, it is rolling and abounds in hills which are often steep but of no great height. Along the lower course of the river the water-shed area is quite flat, particularly in the down-stream portion of the valley of the Concord River (the principal tributary entering the Merrimac within Massachusetts), where large areas of flat meadow and swamp land border upon this stream.

Sanitary Conditions.—At Concord and Manchester in New Hampshire and at Lowell and Lawrence in Massachusetts, large water powers have been developed. These are quite thoroughly utilized by many textile and other industries, in some of which steam is also used as an auxiliary source of power. The tributaries of the river above Lawrence have also been developed for water power at many points, at which mills (the largest of which are at Nashua, N. H.), have been erected and are in operation. From all these places manufacturing refuse and waste of various kinds is turned directly into the stream. Above Lawrence the river receives the unpurified sewage of Concord, Franklin, Laconia, Manchester and Nashua in New Hampshire, and of Fitchburg, Hudson, Leominster and Lowell in Massachusetts, having, in 1900, an aggregate population of 258 751. It further receives the purified effluents from sewage disposal systems in the towns of Clinton,

Concord, Framingham, Marlborough, Natick and Westborough in Massachusetts, having, according to the census of 1900, a total population of 59 118.

In 1878 the Legislature of Massachusetts, by special enactment, made the Merrimac River a free receptacle for sewage, the quantity of which has been greatly increased from year to year, until, in 1900, the amount of such pollution entering the stream directly is estimated to have been 1 gall. in every 40* flowing in driest times in the stream past the city of Lawrence, or 1 gall. in every 200,* representing the average flow of the stream. Referring to the pollution of the Merrimac River, prior to its purification for the water supply of Lawrence, Mr. Hiram F. Mills, as Chairman of the Committee on Water Supply and Sewerage of the Massachusetts State Board of Health, in a communication to the Mayor of the city, dated January 30th, 1891, stated:

"The amount of sewage that has directly entered the river and its branches during the chemical examinations of the past three years (1887–1890), has been estimated to be about 1 gall. in 600 galls. of the river water passing Lawrence."† A comparison of these data indicates that the increase of polluting matters entering the river during the past decade has been very marked.

The total area of the Merrimac River water-shed above the Lawrence Dam is estimated to be 4 630 sq. miles, containing a total population of 480 000,‡ or about 104 persons per square mile, of which figure about 65 persons represent urban population in cities and towns having in 1900 over 5 000 people.

The water power of the stream at Lawrence has been developed and is controlled by the Essex Company, which, from 1845 to 1848, constructed a long dam, averaging 32 ft. in height, across the stream near the center of the city. The mill pond formed by the dam extends up stream to

* These figures, respectively, are based upon the assumption that the Merrimac River receives an amount of crude sewage equivalent to 75 and to 100 galls. per day per person resident on the water-shed above Lawrence in cities provided with sewers which discharge without purification directly into the stream.

† Annual Report, Massachusetts State Board of Health, 1890, p. 533.

‡ This figure has been obtained by using as a basis the estimated population, in 1885, living on the water-shed of the Merrimac River above the dam at Lawrence (Special Report, Massachusetts State Board of Health, 1890, Part I, p. 442). From these data, the population of rural districts on the water-shed in both New Hampshire and Massachusetts was obtained, and the numbers adjusted to the year 1900. The adjustment was accomplished by using the estimated percentage of growth in the entire rural districts of both States during the period from 1885 to 1900. The population of the cities and larger towns upon the water-shed of the river above Lawrence was determined accurately from the census enumeration of 1900. Approximately the same result was obtained by apportioning the 1900 census figures of population of the various cities and towns on the water-shed according to the position of the divide line. The total water-shed population thus obtained is believed to represent quite closely the actual conditions at the present time.

Hunt's Falls, in Lowell, a distance of about 9 miles, and the entire volume of water stored therein is roughly estimated to be equivalent to about 2 hours' flow of the river during times of flood; while, during the driest periods, from 2 to 2½ days may be required to entirely change the water stored in the pond above the dam. The ordinary dry-weather flow of the river, which is comparatively large owing to the very considerable storage obtained in Lake Winnipiseogee and other controlled bodies of water in the lake region of New Hampshire, is estimated to be one-fourth of the average yearly flow, and is probably sufficient to change the water in the pond above the Lawrence Dam in from 30 to 36 hours.

Muddiness of River.—Usually, the water of the Merrimac at the intake of the Lawrence Water-Works does not contain a large amount of suspended matter, such as clay or silt, the normal quantity being estimated to be about 10 parts per million.* During freshets, however, the amount of silt carried by the stream is greatly increased; but this material, being of comparatively coarse structure, requires no great length of time for settlement, so that in a few days after severe storms, the water again assumes its normal character.

In April, 1895, during a period of severe freshet, a study was conducted at the Lawrence Experiment Station to determine the maximum quantity of solids contained in the water of the river at such times, together with the rapidity of settlement and the consequent interval required by the water to return to its normal condition.† On April 16th, during the height of the flood, the water flowing in the river was found to contain total solids to the amount of 1 110 parts per million, of which, 57 parts, or about 5.1%, represented organic matter lost on ignition. A tube 2½ ins. in diameter and 6 ft. long was filled with this water and was allowed to stand quiescent for 24 hours, at the end of which time samples were taken from various depths for analysis, the results of which showed that uniformly in the upper 5 ft. of the tube the water contained from 82 to 83 parts per million of total solids, while in the lower foot the heavier particles, representing nearly 1 030 parts per million, had collected. The percentage of loss on ignition of the solids found in the upper portions of the tube varied from 23 in the first to 30 in the fifth foot from the top.

* Report on the Purification of the Washington (D. C.), Water Supply, 1901; page xiii.

† Annual Report, Massachusetts State Board of Health, 1895, pp. 527–528.

The results of examination of the water flowing in the river on April 16th and on eight succeeding days are given in Table No. 2.

TABLE No. 2.—TOTAL SOLIDS IN MERRIMAC RIVER WATER DURING THE FRESHET OF 1895.

Date.	Total Solids.	Loss on Ignition.	Percentage of loss on ignition.	Percentage of the solids in river water on April 16th.
	Parts per million.			
April 16.....	1 110	57	5	100
" 17.....	452	82	18	41
" 18.....	162	38	24	15
" 19.....	111	27	24	10
" 20.....	78	14	18	7
" 22.....	74	24	32	7
" 23.....	69	16	23	6
" 24.....	53	13	24	5
" 25.....	45	8	18	4

NOTE.—The average Residue on Evaporation, determined on samples of Merrimac River water after passing through filter paper, has been equal to about 40 parts per million. The Total Solids in excess of this quantity probably, therefore, represents, quite closely, Suspended Matter carried by the stream.

By April 25th the river had returned to its usual condition, the total period during which the amount of silt, due to the freshet, was abnormally high being 9 days; while only 3 days were required either for 90% of the solids to settle out of the water, or else for the very turbid water to pass by and water containing only 10% of the maximum amount of mud to take its place.

Table No. 3 has been prepared in order that certain data relative to the water-shed and the character of the water of the Merrimac may be compared with those of other streams in regard to which similar information is available.

CONSIDERATION OF AN IMPROVEMENT OF THE WATER SUPPLY.

Old Filter Gallery.—When the water-works were constructed, from 1873 to 1876, provision was made to take water directly from the Merrimac River through a 36-in. cast-iron pipe, 132 ft. long, beginning at the river, and in extension, through 38 ft. of 4-ft. brick conduit, to a well located under the pumping station. It was also provided that clarified water should be drawn from a stone and brick filter gallery, 8 ft. in width and height, and 300 ft. long, extending down stream from the pumping station at right angles to the direct inlet pipe and conduit. The elevation of the bottom of the conduit, and that of the gallery, is

NOTES TO ACCOMPANY TABLE No. 3.

Merrimac River.—Results were obtained by averaging the records in the reports of the Massachusetts State Board of Health.

Maximum total solids determined on sample collected during height of flood, in April, 1895.

Maximum suspended matter estimated from a comparison of the above figure with the maximum amount of total solids determined on samples after passing through filter paper.

Average suspended matter represents estimate by Board of Experts, in "Report on Purification of Washington (D. C.), Water Supply," 1901, p. xiii.

"Oxygen consumed" is the average for 7 years (1893–1899).

"Total hardness" is the average for 10 years (1890–1899).

"Bacteria" is the average for 7 years (1893–1899).

For remarks on method of analyses, see note, Table No. 17, page 299.

Hudson River.—The urban population in places of 1 000 inhabitants and over is 59; the total was obtained by estimating an increase of 15% in the rural population for the ten years from 1890-1899. See *Transactions*, Am. Soc. C. E., Vol. xliii, p. 248.

Results of examination of the water in the back channel of the river at the intake to the filter plant were kindly furnished by George I. Bailey, M. Am. Soc. C. E., Superintendent of the Albany Water-Works.

Bacterial result represents data from September, 1899, to September, 1900, inclusive.

Potomac River.—Determinations made on river water which had settled about 3.5 days.

See "Report on Feasibility and Propriety of Filtering the Water Supply of Washington, D. C., 1900," pp. 12, 13, 79 and 89.

Bacteria per cubic centimeter represent average of examinations during 8 months (July, 1899, to February, 1900). *Ibid*, pp. 42 and 87.

Estimated average suspended matter in unsettled Potomac River water = 80 parts per million. See "Report on Purification of Washington (D. C.), Water Supply, 1901," p. xiii.

Allegheny and Monongahela Rivers.—Population per square mile represents the estimated urban population only of the principal cities and towns, some of which, however, had not more than 500 inhabitants.

Data compiled from "Report of Filtration Commission," Pittsburg, 1899, pp. 30 to 32, 238 and 271.

Bacterial result for Allegheny River is the average from August, 1897, to January, 1900, inclusive. See *Journal*, New England Water-Works Association, September, 1900, Vol. xv, No. 2, pp. 154 to 156.

Ohio River at Cincinnati.—See "Report on Water Purification," Cincinnati, 1899. pp. 13 and 19. Total water-shed area, including Licking River = 75 700 sq. miles. Figure for population on water-shed includes urban districts only. Averages for "Color" and "Oxygen Consumed" calculated from data on pages 32–36, *Ibid*.

Ohio River at Louisville.—See "Report on Feasibility and Propriety of Filtering the Water Supply of Washington, D. C., 1900," p. 89; from table prepared by George W. Fuller, Assoc. M. Am. Soc. C. E. See also "Report on Water Purification at Louisville, 1898," pp. 15, and 21 to 31.

Figure for population on water-shed includes total inhabitants. The urban population in 220 cities and towns is estimated to be 20 per square mile.

Averages for "Color and Oxygen Consumed" were calculated from data on pp. 21–31. *Ibid*, excluding the months of August and December. Bacterial result is for the period from October, 1895, to July, 1897, inclusive, excluding August to January, 1897, *Ibid*, p. 39.

* This figure, as estimated by Allen Hazen, M. Am. Soc. C. E., on a new basis, is 80. See page 312.

† Total hardness of Potomac River water, as obtained from statement by Mr. R. S. Weston. See page 320.

TABLE No. 3.—Data of Water-Sheds and Average Composition of Water of Certain Representative Rivers in Eastern United States.

Name of river.	Point of measurement and examination.	Water-shed area, in square miles.	Estimated population per square mile of water-shed, 1900.	Chemical Determinations. Parts per million. Average color.	Total solids. Maximum.	Total solids. Average.	Suspended matter. Maximum.	Suspended matter. Average.	Average nitrogen as Ammonia. Free.	Average nitrogen as Ammonia. Alb.	Average nitrogen as Nitrites.	Average nitrogen as Nitrates.	Average oxygen consumed.	Average chlorine.	Average total hardness.	Average bacteria per cubic centimeter.	Period covered by analyses.
Merrimac	Lawrence	4 630	104	0.39	1 110	39	1 050	10	0.052	0.195	0.002	0.079	5.2	2.1	12.4	8 400	1888 to 1899.
Hudson	Albany	8 240	93*	0.37	163	128	72	13	0.112	0.305	8.008	0.054	6.1	9.39†	72.0	27 300	March to September, 1900.
Potomac	Washington	11 040	45	0.20	190	130	100	15	0.010	0.100	0.001	0.700	3.0	3.0	93.0†	3 800	1897 to 1900.
Allegheny	Pittsburg	11 400	24	0.27	806	156	726	42	0.019	0.117	0.000	0.705	.....	21.7	35.5	17 900	August, 1897, to July, 1898.
Monongahela	Pittsburg	7 600	24	0.20	872	208	756	64	0.066	0.149	0.000	0.880	.....	11.8	43.6	19 200	August, 1897, to July, 1898.
Ohio	Cincinnati	72 400	24	0.11	2 556	350	2 333	230	0.025	0.290	0.003	0 600	5.7	10.0	.....	20 000	1898.
Ohio	Louisville	85 000	66	0.18	5 638	500	5 311	350	0.044	0.370	0.008	0.770	7.1	17.0	.....	14 000	1895 to 1897.

about Elevation 26, the same as the floor of the pump-well, or practically 11 ft. below the usual low water in the river. The filter gallery was constructed in a stratum of coarse gravel, with side-walls laid in cement but not thoroughly tight. The bottom consists of cross-blocks of granite, 1 ft. wide and spaced 10 ft. apart, filled in between with small stone. The roof is a brick masonry arch, 8 ins. thick.

It was the original intention to use water from the filter gallery at such times as the river carried undesirable amounts of silt in suspension, although at the time of construction it was uncertain whether or not the full supply for the city could be obtained for all time through this structure. Its inadequacy to meet the requirements of the city was soon apparent, and, in the first annual report of the Water Board, dated January 1st, 1877, it was stated that sufficient water would not percolate through the gravel to the gallery, and that it had become necessary to excavate a channel between this structure and the river. The channel had, at the lower end, a direct open connection with the river, which allowed the water to flow back along the side of the gallery. By this means its capacity was, for a time, apparently increased, though not to the required amount, and a further extension of the works down stream was considered. An enlargement of the channel, in 1878, failed to produce an adequate increase in the yield from the gallery. The report of the Water Board for this year gave evidence of the development of a belief that the importance of a sufficient supply of clear, wholesome water demanded some better mode of filtration than that then in use.

In the eighth annual report, for the year 1883, it was stated that it was apparent that the gallery was gradually silting up, and that most of the water came from the land side. It is evident, therefore, that from the very first, the filter gallery was incapable of furnishing the whole supply for the city, and that its capacity constantly diminished until it was practically insignificant, inasmuch as the flow from the hillside was inconsiderable compared with the demands of the city's consumption.

Various Projects.—Upon several occasions, after the introduction of a public water supply, it was noticed that outbreaks of typhoid fever in Lawrence followed a prevalence of the same disease in Lowell, and in the latter part of 1890 attention was again called with anxiety to the excess of this disease in the city. The citizens, generally, boiled the water for drinking purposes, and many of the schools, mills and dwelling-houses were supplied from springs in the suburbs. On January 31st,

1891, a conference was held between the Water-Works Committee of the City Councils and the past and present members of the Water Board. This meeting resulted in a better understanding of the need of an unpolluted drinking water for the people, but there was a diversity of opinion as to the relative merits of an entirely new and uncontaminated supply from some of the New Hampshire ponds, and the old supply from the Merrimac River, suitably purified by filtration.

About the middle of March, 1891, "The Salem Water Company" was incorporated by the New Hampshire Legislature, with the ostensible purpose of furnishing water to the town of Salem, N. H.; but at the same time there were granted to the company such broad charter powers that other rights to the waters of southern New Hampshire were precluded. Meanwhile, a long series of experiments conducted by the State Board of Health, under the supervision of Mr. Hiram F. Mills, since the establishment of the Experiment Station at Lawrence in 1887, had given evidence that the water of the Merrimac River could be rendered entirely satisfactory and safe for drinking purposes by intermittent sand filtration. The Water Board on June 22d, 1891, presented to the City Councils a report covering the whole question of the improvement of the supply, in which were stated the several objections to a supply from wells, and to a supply from Great Pond in North Andover, or from ponds in southern New Hampshire, and finally recommended that the use of water from the Merrimac River be continued, but that it be filtered—the system of filtration to be determined subsequently. This report was accepted and referred to the Committee on Water-Works, but no other action was taken. About this time a petition was presented to the city government by prominent citizens, and discussed at a joint convention held July 22d, 1891, asking for the appointment of a commission, independent of the Water Board, to consider the entire water supply problem, but no action upon the petition was taken.

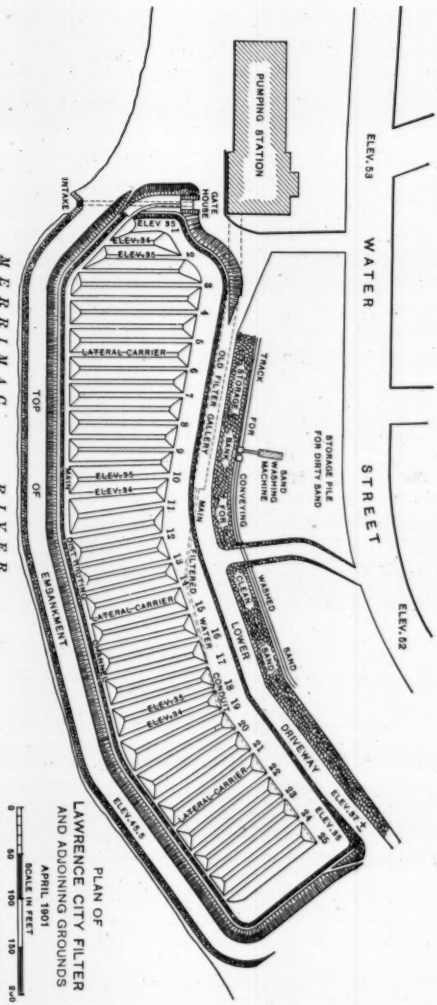
*Advice of State Board of Health.**—The Water Board of 1892, on February 29th, requested the advice of the State Board of Health relative to mechanical filtration. The latter board, on March 9th, stated that it was unable to advise that any of the rapid mechanical filters then in use would purify the Merrimac River water sufficiently to insure exemption from typhoid fever, but they did advise, however, that by a process of slow intermittent filtration at a rate of 400 000 galls. per acre per day, bac-

* The information in this section has been obtained from the Reports of the Massachusetts State Board of Health.

teria could be very completely removed. On June 5th, 1892, the State Board of Health, in answer to a subsequent communication, dated May 6th, relative to filtration through sand, advised the Water Board that a satisfactory purification of river water would be obtained with sand filters of proper material 5 ft. deep, operated at a rate of 1 500 000 galls. per acre daily. It was also stated that, in order to produce satisfactory results, it was deemed necessary to free the river water from the sediment carried during freshets, before applying it to the filters, which could be located conveniently upon the hill just above the reservoir. To accomplish the preliminary clarification of the water, it was advised that a filter of coarse sand should be provided at the river bank. The Board of Health also advised that, in its judgment, a marked decrease in the death rate from typhoid fever could be brought about simply by the construction and proper maintenance of the filter referred to, at the river bank. Furthermore, it was strongly recommended that the excessive death rate from typhoid fever demanded that some method of purifying the water be adopted at once.

Filter Construction.—The Water Board concluded that the project to build a filter at the river bank was the more feasible, and decided to proceed with its construction, according to plans and directions to be given by the State Board of Health. On August 4th, 1892, a visit of the State Board of Health, the Water Board and city officials was made to the proposed site. The City Councils granted the Water Board permission directly afterward to expend $20 000 of its earnings for the purpose of filter construction. Ground was broken on September 10th, and 150 men placed upon the work. On November 13th, an additional $10 000 was allowed for its further prosecution, and this sum was found sufficient until the end of the year.

At the beginning of the new year, owing to a feeling of uncertainty on the part of the City Councils as to the promised action of the filter in purifying the river water, there was considerable discussion in regard to renewing the work of construction. However, it was finally decided on February 13th, 1893, to proceed with diligence, under the joint direction of the Water-Works Committee of the City Councils and the Water Board, the cost being provided from the Incidental Fund. After October 1st, the expenditures were again charged to the water-works account. The filter was completed and water turned on Sep-



PLAN OF
LAWRENCE CITY FILTER
AND ADJOINING GROUNDS
APRIL 1901
SCALE IN FEET
0 50 100 150 200
COPIED FROM PLAN AND DATA FURNISHED BY A. D. MARBLE, CITY ENGINEER.

Fig. 1.

tember 20th, 1893, and it has continued to furnish the city with filtered water to the present time.

The cost of construction of the filter to the end of the year 1893 was about $65 500. Additional construction, for the most part undertaken with a view to facilitate operations and render the maintenance more efficient and less expensive, has been carried out from time to time, the cost of which appears in detail in a later portion of this paper.

Description of the Filter.*

The Lawrence City Filter was designed by Mr. Hiram F. Mills, Chairman of the Committee on Water Supply and Sewerage of the State Board of Health of Massachusetts, to whom credit is due for giving his time and energy to bring about such a benefit to the city.

Location and Dimensions.—The filter is located, as shown in Fig. 1, on the northerly bank of the Merrimac River, about one mile above the Essex Company's Dam, and immediately east of the pumping station.

It is somewhat irregular in outline, having a total length of about 750 ft , an average width of 140 ft., and a surface area, including the main carrier, of 2.44 acres. The filter surface is divided by transverse carriers into 25 portions or beds, each having an average width of 30 ft. This gives an available filtering surface, including lateral carriers, of 2.36 acres; or, without these carriers, of about 2.3 acres.

In constructing the filter, the bed and bank of the river were excavated generally to 7 ft. below usual low water (equal to Elevation 37, which is the top of the flash boards on the Essex Company's dam), for a distance of about 150 ft. southerly from the old filter gallery, and in the river bank east, or down stream, a sufficient distance for the total length of the filter. Suitable material from the excavation was placed in a tight embankment on the ends and on the side toward the river, of such a height as to be 9 ft. above usual low water, and high enough to exclude freshets.

Main Conduit and Underdrains.—The underdrain pipes which extend across the filter from north to south, were arranged with centers about 30 ft. apart. For a width of 5 ft. along the line of drain pipe the exca-

* This description is largely abstracted from the paper by the designer, entitled "The Filter of the Water Supply of the City of Lawrence, and Its Results;" to which the writers refer for further details not given here. (See 25th Annual Report of Massachusetts State Board of Health for 1893, pp. 545-560.)

vation was continued to Elevation 29, and midway between the lines of underdrains, for a width of 5 ft., the depth of the excavation was only to Elevation 31. The bottom between these two levels, for a width of 20 ft., was excavated with a slope of 1 in 10. Thus it will be seen that the bottom of the filter when excavated, ready to receive the filtering material, was waved, with alternate level ridges between the lines of underdrains, and flat valleys in which the underdrains themselves were placed.

The old filter gallery extended easterly along the northerly edge of the excavation for the filter, from the pumping station at the westerly or up-stream end, through a distance of 300 ft. to a point about opposite Bed No. 11. From the gallery a main conduit was extended eastward in the filter beds, running parallel with and about 20 ft. south of the northerly edge of the filter. This conduit is circular in shape and its westerly portion is 4 ins. thick, and built of brick laid without mortar in the end joints, thus allowing spaces a little more than $\frac{1}{8}$ in. wide for the entrance of filtered water. The conduit is 2 ft. in diameter for 105 ft. from the end of the filter gallery; thence 20 ins. for 120 ft.; 16 ins. for 65 ft.; 12 ins. for 30 ft.; continuing with successive lengths of 30 ft. each of vitrified pipes having diameters of 10, 8 and 6 ins., respectively. The entire conduit was surrounded with 8 ins. of gravel, the inner 4 ins. being of stones about 2 ins. in diameter, and this was covered with layers of stones of decreasing sizes.

From the conduit and old filter gallery, which was pierced at suitable places for the purpose, lateral underdrains of vitrified pipe were laid in a southerly direction upon the excavated bottom, with a slope of 1 in 100, for a distance equal to four-tenths of that from the main conduit to the southerly edge of the filter. Beginning at the main conduit, the pipes were laid in the following order: One length of 10-in., two lengths of 8-in., from 35 to 65 ft. of 6-in., ending with three lengths of 4-in. The pipes were placed so that the spigot end of one approached, but did not enter, the bell end of the next, and the drains throughout were covered by a 4-in. layer of 2-in. gravel. Upon this layer, which averaged about 6 ins. in actual depth, there were placed successive courses of gravel of the following sizes and of the approximate thicknesses (given for an average section) to make a total of 12 ins. of underdrain material: 2 ins. of $1\frac{1}{2}$-in., 1 in. of $\frac{3}{4}$-in., 1 in. of $\frac{3}{8}$-in., 1 in. of $\frac{3}{16}$-in. gravel, then 1 in. of coarse mortar sand. The course of

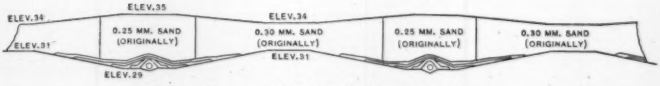
$\frac{3}{16}$-in. material was spread out to a width of 17 ft., and the last course was spread out to a width of 20 ft. Beyond the pipes, the largest stones, forming the lowest course, were laid upon the surface of the excavation and were spread out to a general width of 5 ft., with the next courses each spread out a little beyond the one just below. The $1\frac{1}{2}$-in. gravel extended southerly to within 12.5 ft. of the edge of the main distributing channel, with the end of the coarse sand 7.5 ft. further south.

The largest stones used in the underdrains were selected by hand, and, if covered with dust or dirt, were washed. The smaller sizes were obtained by screening, but were not washed. The stones of the two larger sizes, namely, 2-in. and $1\frac{1}{2}$-in., were placed by hand at the open joints of the pipes, but the layers of smaller sizes were all shoveled into place. The arrangement of all the above-mentioned material, including the sand, is shown for several sections in Fig. 2.

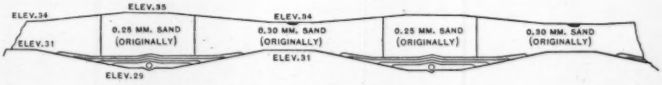
Filter Sands.—Immediately over the center lines of the underdrains, that is, over that portion of the beds excavated to the greatest depth, and extending 5 ft. on either side, there was placed a body of sand, with a maximum depth of 5 ft. in the center. Over the ridges of the excavation for the filter, midway between the lines of underdrains and for a distance of 10 ft. on either side, there was placed a body of sand, having a minimum depth, over the flat portion of the ridge, of 3 ft. The effective size of the sand over the underdrains was approximately 0.25 mm. and of the remainder was about 0.30 mm.

It will be seen by this construction that the lowest portions of the filter surface are those directly over the ridges of the excavation and half way between the underdrains; the general elevation of the surface of these parts being 3 ft. below the usual low-water elevation of the river, and 1 ft. below the sand surface over the underdrains. For a width of 5 ft. over the center lines of both ridges and underdrains the surface of the sand is flat, and between these level areas the sand has a slope of 1 in 10. This form of surface is such that water can be readily introduced upon the filter to refill, without producing currents sufficiently rapid to cause appreciable disturbance.

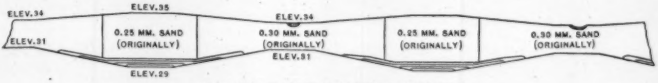
Sand and gravel for the filter were obtained from a large deposit back from the river, and distant about $\frac{7}{8}$ mile from the works. The material found at this place was by no means uniform, and suitable grades were obtained by selecting certain layers and pockets, and by judicious screening and mixing of these various kinds.



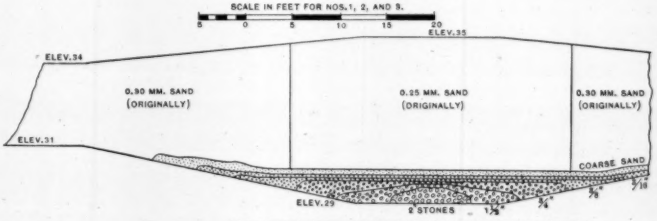
No. 1. CROSS-SECTION AT NORTH END OF BED.



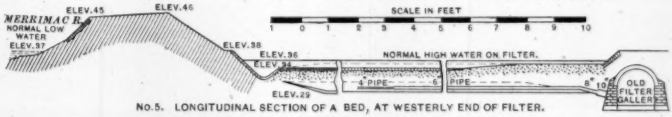
No. 2. CROSS-SECTION AT BEGINNING OF PIPE UNDERDRAIN.



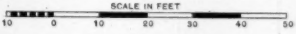
No. 3. CROSS-SECTION AT SOUTH END OF BED.



No. 4. CROSS-SECTION AT END OF LOWEST GRAVEL UNDERDRAIN.



No. 5. LONGITUDINAL SECTION OF A BED, AT WESTERLY END OF FILTER.



TYPICAL SECTIONS OF UNIT BEDS IN LAWRENCE CITY FILTER.

APRIL, 1901.

COPIED FROM PLAN FURNISHED BY A. D. MARBLE, CITY ENGINEER.

FIG. 2.

Care was taken in depositing the sand in the filter to render the mass as homogeneous as possible. It was placed in two layers. The first layer was covered with planks as fast as deposited, and, upon the plank platform, loads were hauled and dumped near the edge, so that a part fell naturally into place, and the remainder was shoveled. Material, exposed to sun and wind before being finally placed, was shoveled over, to lessen the danger from the collection of the larger particles at the bottom of dry piles of sand. The second layer was dumped upon platforms resting on the first and then shoveled to position, the compacted sand in the first layer beneath the planking being first spaded to a depth of 1½ ft. before the second layer was placed.

Intake and System of Distribution.—The direct conduit, by which water was formerly supplied from the river to the pumping station, is at the westerly or up-stream end of the filter, and the flow through it was controlled by a gate at the inner end. About 40 ft. southerly from the engine-house, the top of this conduit was pierced and two cast-iron pipes were set in a chimney built at this opening. The pipes are 2 ft in diameter, and each has a gate, the center of which is 2 ft. below the usual low water in the river. The use of one of these pipes to supply a possible future extension of the works west of, or up-stream from, the present filter has been suggested.

The other pipe discharges into the main carrier, or distributing channel, which extends along the entire southerly edge of the filter to a point within about 35 ft. of the easterly end. The edge of this channel toward the filter remains level throughout its length, the elevation of this edge corresponding with the uniform elevation of the sand surface of the southerly edge of the filter, or 3 ft. below usual low water in the river. At the upper or westerly end, this carrier has a semicircular section of 3 ft. radius, but as it extends easterly, the width and depth constantly decrease, so that at the lower end its width is reduced to 2 ft., and the bottom is only 6 ins. below the level of the inner edge.

In each of the level valleys of the sand surface a shallow distribution carrier extends from the main channel to a point 32 ft. from the opposite or northerly side of the filter. At the main channel, each lateral carrier is 2 ft. wide and 3 ins. deep, decreasing to a width of 1 ft. and a depth of 1 in. at the northerly end. The edges of these distributors are level and at the same elevation as the inner edge of the main carrier.

The main carrier is constructed of cobble paving stones, from 4 to 6 ins. in diameter, laid in Portland cement mortar on a gravel bed 8 ins. thick. The lateral carriers are made of cobble stones, 4 ins. in diameter, laid in Portland cement mortar directly on the sand surface. The interior surface of all these channels was made smooth by a coat of mortar.

Description and Results of Operations.

Methods in General.—From the time when the filter was first started it has been customary to fill from above over the surface. The water from the river is turned on gradually at the gate-house at the westerly end of the filter, allowed to pass out into the main channel, and from this along the lateral carriers to the different portions of the filter. At first the water settles into the sand nearest the carriers, but at length rises slowly and covers the entire sand surface. When a depth of from 1½ to 2 ft. of water has been obtained over the carriers the pump at the station is started. The more slowly the whole process is conducted the smaller will be the amount of entrained air in the filter, and the initial loss of head correspondingly less. At various times of urgency undue haste in refilling has actually caused loss of time, inasmuch as the amount of air retained in the bed has resulted in creating excessive losses of head shortly after starting, causing a reduced rate of filtration below that which might have been obtained with more deliberate action.

Almost without exception the water in the river has been high enough to flow upon the filter by gravity and maintain the water surface at a suitable elevation; but upon one occasion, in January, 1900, for a period of about 10 days, the water in the river was approximately at Elevation 36, and it was necessary to resort to pumping to supply water to the filter. Two centrifugal pumps of 8-in. and 6-in. discharge were used. The former was purchased and housed permanently for regular use in draining the filter, as well as to serve as a safeguard in case of unusually low water in the future.

In operating the filter it has been customary to maintain the water surface upon the beds at Elevation 36, or 1 ft. below the usual elevation of low water in the Merrimac River. This is equivalent to a depth of 2 ft. over the lateral distributing carriers, and of 1 ft. over the crowns of the beds when at grade. During the coldest weather, when it has not

been possible to scrape the surface, the water has frequently been raised to Elevation 37, and at less frequent intervals to Elevation 37.50 in order to force the required amount through the filter.

Whenever the filter is so clogged that the maximum loss of head will not force a sufficient quantity through it, the filter is shut down and drained in the following manner: The inlet gate is closed, preventing the further entrance of river water upon the filter, and pumping is continued at a constantly decreasing rate until the water already on the beds passes into the sand and below the surface to such a depth that laborers can work upon it. Since January, 1900, this operation has been greatly facilitated by the use of the 8-in. centrifugal pump provided with a double suction and discharge, so arranged as to force water from the filter to the river, or in the reverse direction. The clogged surface over those beds which require cleaning is then scraped with shovels. The scrapings are carried by wheelbarrows to the driveway, the surface is raked and smoothed, and the filter again put in service. Later, the dirty sand left upon the driveway is removed to a bank near the washer, from which, after washing, it is dumped over a paved slope, the foot of which extends to the lower driveway, ready for replacing upon the filter at the time of sanding. This work constitutes the general routine at the filter, but it is constantly interrupted during the winter season by the ever-recurring necessity of removing snow and ice from the filter surface. The ice is never allowed to become so thick that it will touch the sand surface during high water upon the filter, and it is either cut and thrown out, or large fields floated away from such beds as need scraping.

For purposes of reference the beds of the filter are numbered from 1 to 25, beginning at the westerly end.

Quantitative Results.—The amount of water pumped, and thus the amount filtered, has been dependent upon the demands of the city's use, as shown by the draft upon the reservoir. Whenever the surface of the filter is not excessively clogged, the yield is generally constant, and the rate about 5 000 000 galls. per day, the required quantity being obtained by running the pumps the necessary number of hours. As frequently some time is lost between stopping the low and starting the high-service pumps, and *vice versa*, and also because the filter is often out of service for scraping or similar requirements,

there are long intervals when there is almost constant need of pumping continuously, except occasionally upon Sundays.

TABLE No. 4.—AVERAGE DAILY QUANTITY OF WATER PUMPED, IN MILLIONS OF GALLONS, AND AVERAGE HOURS OF PUMPING PER DAY, AT LAWRENCE, MASS., FOR EACH MONTH, 1893–1900.

Month.	1893.		1894.		1895.		1896.		1897.		1898.		1899.		1900.	
	Quantity.	Hours per day.	Quantity.	Hours per day.	Quantity.	Hours per day.	Quantity.	Hours per day.	Quantity.	Hours per day.	Quantity.	Hours per day.	Quantity.	Hours per day.	Quantity.	Hours per day.
January	4.0	20.2	2.9	15.0	2.9	15.6	3.1	16.2	3.4	18.9	3.4	25.2	3.2	19.3	3.3	22.1
February	3.7	18.6	2.8	17.0	3.0	16.9	3.1	16.8	3.1	23.7	3.1	30.3	3.3	19.3	3.3	24.6
March	3.2	16.3	2.7	14.6	2.9	18.0	2.8	17.4	2.6	18.0	3.4	28.2	3.0	21.0	3.4	25.3
April	2.7	13.5	2.4	13.2	2.6	16.1	2.8	15.6	2.8	21.2	3.3	19.6	3.0	19.2	3.0	33.9
May	2.9	15.2	2.8	14.2	2.8	14.9	3.2	17.7	2.9	21.7	2.9	17.5	3.2	19.7	3.2	29.5
June	3.0	16.1	3.0	15.8	3.1	16.8	3.2	17.3	3.0	17.8	3.3	26.7	3.5	21.4	3.6	23.1
July	3.0	15.8	3.0	15.3	2.9	15.3	3.1	16.3	3.4	20.1	3.3	29.8	3.5	20.5	3.7	22.9
August	2.6	13.8	2.8	14.2	2.8	15.1	3.0	15.6	3.1	18.1	3.2	21.5	3.3	18.0	3.6	20.8
September	2.8	14.7	2.6	13.7	2.9	15.1	3.2	16.7	3.3	18.7	3.8	22.7	3.5	19.8	3.5	22.4
October	2.7	13.9	2.7	13.8	2.8	15.0	2.8	15.4	3.2	19.1	3.5	23.7	3.1	18.8	3.3	19.9
November	2.6	13.6	2.6	13.8	2.7	14.9	2.8	15.6	3.1	19.8	3.7	23.6	3.0	19.2	3.0	18.1
December	2.7	13.9	2.5	13.4	2.7	14.0	3.0	15.7	3.1	22.3	3.0	17.9	3.0	19.5	2.9	18.0
Average	3.0	15.5	2.7	14.5	2.8	15.6	3.0	16.4	3.1	19.9	3.3	23.9	3.2	19.6	3.3	23.4

NOTES.—The high-service pump was put into service in December, 1896, and since that date the quantities include the water used in both low and high service.

The average hours of pumping are based upon the total for each month divided by the total number of days, and, after 1896, include the time for both the high and low-service engines.

In order to present this information in comprehensive form, there is given in Table No. 4 the average daily consumption by months (both high and low-service being included, since 1896), and the average number of hours per day during which the pumps were operated. The hours of pumping per day represent the totals per month divided by the total number of days, and, since 1896, is given as the sum of the hours of operation of both the high and low-service pumps. With the exception of portions of the years 1898 and 1900, when a small-capacity, low-duty pump was used for some of the time upon the high service, and which of necessity was run very often when the low-service engine was in operation, the hours given represent quite closely the time during which it was necessary to operate the filter.

The relation of temperature to the consumption of water is so intimate that it has been deemed advisable to present in Table No. 5 statistics of the temperature of the air at Lawrence during the past eight years. These records were obtained from the reports of the

United States Weather Bureau, and are the results of observations made by the Essex Company at a station located on the north side of the Merrimac River near the dam, in an unexposed but not particularly well-sheltered place.

TABLE No. 5.—AVERAGE MONTHLY TEMPERATURE OF AIR AT LAWRENCE, MASS., ALSO THE MAXIMUM AND MINIMUM FOR EACH MONTH, 1893–1900.

Months.	1893.			1894.			1895.			1896.			1897.			1898.			1899.			1900.		
	Aver.	Max.	Min.	Aver.	Max.	Min.	Aver.	Max.	Min.	Aver.	Max.	Min.	Aver.	Max.	Min.	Aver.	Max.	Min.	Aver.	Max.	Min.	Aver.	Max.	Min.
January	17	51	—11	26	53	— 2	22	45	— 3	21	42	—14	25	56	— 3	25	51	—13	25	55	— 9	26	54	—13
February	24	52	— 5	22	49	—12	19	46	—11	26	54	—15	27	51	— 5	28	57	—19	23	51	—14	27	59	— 5
March	32	54	5	40	74	15	32	49	7	30	60	8	34	58	0	40	64	18	32	59	11	31	56	5
April	44	70	21	47	77	17	45	76	22	48	87	22	48	84	17	43	72	19	46	83	25	45	80	19
May	57	87	34	59	85	34	61	94	30	62	96	33	58	85	34	57	84	33	58	90	32	54	93	28
June	66	92	50	69	93	44	70	95	46	67	96	46	62	91	44	66	91	45	69	93	49	68	92	39
July	71	90	52	74	97	53	70	92	52	73	93	52	75	100	52	72	97	50	72	92	50	73	95	47
August	69	90	48	69	89	47	71	94	46	71	97	47	71	90	54	72	91	50	69	90	49	70	96	49
September	59	81	39	65	86	39	64	94	39	61	86	37	62	94	38	65	91	41	61	82	37	64	90	38
October	54	77	26	52	74	34	47	69	22	48	72	27	53	86	26	53	82	31	53	77	28	55	83	26
November	39	63	12	35	60	10	42	73	15	43	71	16	38	66	8	40	61	18	39	63	21	42	70	14
December	26	53	—10	27	49	— 2	31	61	4	25	52	— 4	31	60	0	27	54	—14	32	63	2	29	56	2
Average	46	..	....	49	..	....	48	..	....	48	..	....	49	..	....	49	..	....	48	..	....	49	..	....
Maximum	..	92	....	..	97	....	..	95	....	..	97	....	..	100	....	..	97	....	..	98	....	..	96	....
Minimum	..	..	—11	..	..	—12	..	..	—11	..	..	—15	..	..	— 5	..	..	—19	..	..	—14	..	..	—13

NOTE.—All data in this table are compiled from the records of the United States Weather Bureau.

Quantities Between Scrapings.—The importance of these data in regard to any filter is generally recognized, but in this particular case, owing to the exigencies of operation, the information is less definite and valuable than it would be if the yield from single beds was measured. In the case at hand, while at the time of scraping the whole surface is necessarily drained, different beds may be and usually are cleaned each time, also the same order of scraping the beds is rarely observed twice in succession.

It has been possible to select a few cases where the whole area of 25 beds was completely scraped in 1 or 2 days, and when no scraping occurred on intermediate dates. These results, representing the longest periods and greatest quantities passing through the 2.36 acres of filtering surface between two successive complete scrapings, are given in Table No. 6. The table also indicates that in actual practice the filter has not been operated during recent years more than a month without cleaning, and, generally, the period has been less.

TABLE No. 6.—LONGEST PERIODS BETWEEN COMPLETE SCRAPINGS OF THE LAWRENCE FILTER FOR THE YEARS 1893–1900.

Number.	Last day of a complete scraping.	First day of next complete scraping.	Length of period, in days.	Total quantity between scrapings, in millions of gallons.
1	March 26th, 1898	April 27th, 1898	32	109
2	April 28th, "	May 27th, "	29	90
3	May 28th, "	June 23d, "	26	80
4	June 24th, "	July 14th, "	20	74
5	July 15th, "	August 9th, "	25	85
6	August 11th, "	" 30th, "	19	67
7	" 31st, "	September 22d, "	22	85
8	April 6th, 1899	April 26th, 1899	20	71
8	June 14th, "	July 6th, "	22	78
10	July 7th, "	" 20th, "	13	49
11	April 18th, 1901	May 30th, 1901	41	121

Record of Scrapings.—There is a lack of definite data in regard to the exact dates of scraping prior to the spring of 1895, and of the dates and amount of sanding prior to 1897. Information has been culled from various sources, such as the regular scraping record books, time books, pumping station records, diaries, miscellaneous note books, etc., and it is believed that the following statements and tables contain as exact information as will likely ever be obtained.

It appears that, after starting the filter upon September 20th, 1893, the whole surface was scraped twice before December 1st. Apparently, the filter was not cleaned again until the next spring. The work began about the middle of March and was completed about the middle of April. The surface was again scraped during the latter part of the month of July. The best evidence goes to show that the entire filter was again cleaned between the latter part of August and the early part of October. The whole surface was again gone over during the last half of the month of November. To summarize, it will be seen that the filter was scraped four times during the year 1894, although it is possible that a few of the beds at the upstream end were scraped at another time, of which there is no memorandum.

The record for the first three months of 1895 is somewhat indefinite, but it is believed that there is sufficient information to warrant including the record for the year in the tables. The data for each of the years 1895 to 1900 are given in Tables Nos. 7 and 8, in the form of total number of times of scraping for each bed and totals for each

month. Occasionally, beds have been cleaned in portions, or at flat ends only, but each such scraping is given the proper fractional weight in making up the totals, the latter being recorded in each case, however, to the nearest unit.

TABLE No. 7.—Record of Scrapings of the Lawrence Filter, Arranged by Beds for each Year, 1895–1900.

Beds.	1895.	1896.	1897.	1898.	1899.	1900.
1	15	16	17	14	17	16
2	16	17	17	14	17	16
3	16	17	18	14	17	16
4	16	16	19	14	16	16
5	16	16	19	14	16	16
6	16	15	18	14	16	15
7	17	16	18	14	16	16
8	17	14	16	14	16	16
9	16	14	16	15	15	16
10	16	14	16	15	17	16
11	13	13	17	15	16	17
12	14	13	18	15	18	18
13	14	12	18	15	18	16
14	13	11	18	14	19	16
15	13	10	17	12	19	14
16	13	11	16	13	19	15
17	13	10	16	13	19	15
18	13	10	16	12	17	15
19	12	11	17	11	16	16
20	12	10	16	12	16	16
21	11	9	14	13	17	16
22	11	8	15	12	17	16
23	11	7	14	10	16	16
24	10	7	15	10	16	15
25	10	7	15	12	15	15
Totals	344	304	416	331	421	395
Averages	14	12	17	13	17	16

TABLE No. 8.—Record of Scrapings of the Lawrence Filter, Arranged by Months for the Years 1895–1900.

Months.	1895.	1896.	1897.	1898.	1899.	1900.
January	20	9	20	17	25	25
February	20	20	34	22	12	25
March	20	43	46	34	6	25
April	27	31	50	25	49	33
May	30	31	25	25	26	47
June	25	18	25	25	30	27
July	29	32	39	25	50	40
August	31	24	32	47	46	35
September	25	20	27	28	31	48
October	39	16	32	40	43	26
November	53	46	53	25	53	39
December	25	14	33	18	50	25
Totals	344	304	416	331	421	395
Averages	29	25	35	28	35	33

Record of Sandings.—The first sanding of the filter was done after some of the scrapings in 1894, and, as the washing machine for cleansing dirty sand was not installed until the summer of 1895, it is probable that unused sand, obtained during construction, and an additional amount hauled from Gale's Hill, was used during these years. That replaced in 1896 and thereafter was, for the most part, washed sand, which had been removed from the filter surface, and the sanding was done in bulk.

In 1897 the first record of the dates of sanding appears, together with the depths replaced at the crowns of the beds, the sides being filled slightly to bring them to the level of the lateral carriers. The depths of sandings of each bed at the crown for the years 1897, 1899 and 1900 are given in Table No. 9.

TABLE No. 9.—Depth of Sand in Inches Replaced at the Crown of each Bed of the Lawrence Filter for the Years 1897, 1899 and 1900.

Beds.	1897.	1899.	1900.
1	13.6	11.9	9.8
2	13.6	14.0	10.6
3	13.0	12.9	12.0
4	16.0	25.8	12.7
5	15.4	22.5	12.0
6	15.4	21.5	12.5
7	15.2	21.8	11.0
8	12.5	34.2	10.8
9	15.1	24.0	12.0
10	10.5	25.2	11.9
11	10.0	20.8	11.4
12	11.1	18.0	9.5
13	10.7	15.0	13.5
14	10.9	15.5	13.5
15	9.9	11.5	18.3
16	10.2	12.0	18.5
17	9.2	9.1	16.5
18	11.3	12.0	15.3
19	9.5	9.2	15.0
20	9.3	15.4	22.2
21	8.7	9.5	15.0
22	8.4	9.8	18.3
23	9.5	16.0	25.0
24	8.2	15.9	24.0
25	9.5	14.0	18.0
Averages	11.5	16.3	14.8

In 1898 extensive experiments were made during the summer months, and the latter portion of the year was occupied in the repair work on the underdrains in certain beds, so that very little sanding was done.

Beds Nos. 1, 2, 3 and 12 were filled to a depth of about 13 ins., and the beds repaired (Nos. 13 to 22, excepting No. 20) were brought to original grade.

From the summaries in the tables it will be seen that, on an average, each bed has been scraped about 15 times per year, and because the average of depths of sand replaced at the crowns is equivalent to about three-fourths of this amount over the entire filter area, the average depth removed at each scraping has been practically $\frac{3}{4}$ in. The average depth replaced at the crowns, in 1899 and 1900, was $15\frac{1}{2}$ ins., equivalent to a depth of $11\frac{1}{2}$ ins. over the whole surface, or a total of about 3 500 cu. yds.; which figure agrees approximately with rough measurements of the amount of material removed and replaced each year.

Loss of Head.—The loss of head is the difference between the level of the water on the filter surface and that of the surface of the water in the pump-well, the latter elevation being indicated by a counterweight sliding on a graduated rod in the engine-room above and showing the position of the float below, referred to the datum of levels. Formerly, the elevation of the water on the filter was read directly upon a gauge board, but within recent years it also has been indicated by the position of a counterweight moved by a float and sliding along a scale board in the gate-house.

The bottom of the pump-well is at Elevation 26, and the bottoms of the outlets of the underdrains in the filter are at Elevation 29; thus it is evident that the difference of the two levels mentioned above indicates an incorrect loss of head whenever the water in the pump-well is drained below Elevation 29. Prior to September, 1898, the losses of head were frequently recorded greater than could have been possible. It is probable that, because of this drawing down, air was admitted backward into the underdrains, and a greater resistance was occasioned than would have occurred if the water in the pump-well had been kept above Elevation 29. However, in compiling the data in Table No. 10, the maximum possible loss of head was assumed to have been that which would have obtained with the water at Elevation 29 as it passed out of the underdrains.

Table No. 10 contains a statement of the average losses of head by months, for the seven complete years of operation; together with the average speed of the pumps and the average temperature of the river water as applied to the filter.

TABLE No. 10.—AVERAGE LOSS OF HEAD IN FEET IN THE LAWRENCE FILTER, BY MONTHS, FOR THE YEARS 1894–1900; ALSO AVERAGE REVOLUTIONS OF THE LOW-SERVICE PUMP PER MINUTE AND THE AVERAGE TEMPERATURE OF THE RIVER WATER IN DEGREES FAHRENHEIT.

Months.	1894.			1895.			1896.			1897.			1898.			1899.			1900.		
	Loss.	Rev.	Temp.	Loss.	Rev.	Temp.	Loss.	Rev.	Temp.	Loss.	Rev.	Temp.	Loss.	Rev.	Temp.	Loss.	Rev.	Temp.	Loss.	Rev.	Temp.
January...	3.1	16	32	6.2	15	32	5.8	15	34	7.2	15	32	8.0	13	32	4.4	15	33	6.0	13	33
February.	6.0	14	32	6.9	15	32	5.9	16	34	7.5	13	32	8.0	10	33	4.6	15	33	6.2	13	33
March.....	5.4	15	38	7.6	13	32	7.2	14	35	7.6	13	33	7.7	12	38	5.5	12	33	6.0	14	34
April.....	4.2	15	48	7.1	13	43	5.8	15	46	6.9	14	45	5.8	15	45	5.0	14	41	5.7	14	44
May.....	5.0	16	58	5.5	15	61	5.6	15	63	4.9	16	60	6.1	15	57	4.3	14	59	5.3	14	56
June.....	4.6	15	69	5.0	16	73	5.2	15	69	5.0	16	65	4.5	15	70	3.6	15	73	3.8	15	72
July.....	3.3	16	78	4.4	16	74	3.6	16	77	5.0	16	74	4.7	16	77	3.6	15	77	3.9	15	78
August...	3.6	16	75	4.4	16	74	3.2	16	77	5.2	15	73	5.5	14	76	3.7	16	75	3.7	15	77
September	4.1	16	72	3.4	16	68	3.8	16	68	5.0	16	68	4.6	15	70	3.6	16	68	3.7	15	72
October...	4.5	16	56	5.5	15	50	5.4	15	52	5.5	16	57	4.9	13	58	4.0	15	57	4.0	15	61
November	5.8	16	40	5.8	15	42	6.3	15	42	6.9	14	42	5.7	14	42	4.6	14	42	4.4	15	46
December.	5.9	16	32	5.3	16	35	6.1	15	33	7.3	13	34	4.7	15	33	5.1	14	36	5.1	15	34
Yearly average..	4.7	15.6	52	5.6	15.1	51	5.3	15.3	52	6.2	14.8	51	5.9	13.9	53	4.3	14.6	52	4.8	14.4	53
Average, Dec.-Mar.	4.8	....	34	6.6	....	32	6.0	....	35	7.1	....	33	7.7	....	34	4.8	....	33	5.8	....	34
Average, Jun.-Sep.	3.9	....	74	4.3	....	72	3.9	....	73	5.0	....	70	4.8	...	73	3.6	....	73	3.8	....	75
Maximum	7.7	....	...	8.0	....	80	8.0	....	84	8.5	....	82	8.5	....	82	6.8	....	80	6.7	....	82
Minimum.	1.8	....	32	2.6	....	32	2.7	....	32	3.2	....	32	2.4	....	32	2.5	....	32	3.1	....	32

NOTE.—Prior to May, 1897, the records do not show the elevation of water in pump-well when pumping. For those days during this time, when the loss of head is greater than 7.0 ft., and for those days afterwards when the water in the pump-well is below Elevation 29.00, the maximum is assumed to be the difference between the elevation of the water on the filter surface and Elevation 29.00.

The table exhibits clearly the greater loss of head under which the filter has been operated during cold weather, and the fact that, even with the maximum possible head, the rate of pumping was necessarily reduced, especially during the winter of 1897–98.. The relative improvement in these factors during 1899–1900 is also noticeable.

Effect of Scraping upon Quantitative Efficiency.—It is customary to scrape a filter when the amount of water which comes through with the maximum loss of head has diminished below that required, and it is expected that with a clean surface the normal quantity will be obtained with a smaller head than before, but at Lawrence this result has not always been attained. Occasionally, it has seemed that air has been caught in the body of the filter when refilling, and its presence has

TABLE No. 11.—Effect of Scraping Five or More Beds of the Lawrence Filter, as shown by the Change in the Loss of Head.

Date of scraping.	Beds Scraped.		Average Loss of Head For Four Days.*		Percentage. Gain in loss of head.
	Number.	Designation.	Before scraping.	After scraping. (Adjusted.)	
(1)	(2)	(3)	(4)	(5)	(6)
1895.					
June 19	10	1-10	6.8	4.4	35
June 26-27	15	11-25	4.0	4.4	—†
July 16-18	15	11-25	4.4	4.2	5
September 10-12	17	9-25	3.3	3.0	9
1896.					
April 8-10	25	1-25	5.3	6.8	—
May 27-29	15	5-12, 17-23	6.4	4.6	28
June 6-8	14	1-14	4.9	3.3	33
August 14	8	1-8	3.6	2.8	22
November 10-13	19	1-10, 16-24	6.5	5.5	15
November 24-25	16	10-25	6.2	6.2	—
December 10	9	1-9	6.1	6.3	—
December 31	5	6-10	6.0	6.8	—
1897.					
May 5-6	11	15-25	4.9	5.2	—
May 19	6	1-6	4.9	4.7	4
May 28	8	7-14	3.8	5.3	—
June 22-25	21	5-25	6.5	4.6	29
July 12-14	17	1-17	5.1	3.8	25
August 17-19	14	1-8, 20-25	5.8	5.4	7
August 25-27	11	9-19	5.1	6.1	—
September 15-16	12	5-16	4.9	4.5	8
October 5-7	10	1-10	5.8	6.1	—
October 13-14	15	11-25	5.4	4.8	11
October 26-28	7	1-7	4.1	6.6	—
November 10-11	16	10-25	7.0	6.8	3
December 1-2	6	10-15	7.0	6.7	4
December 11-14	25	1-25	7.0	7.2	—
1898.					
February 26	9	1-9	8.0	8.5	—
March 25-26	23	1-22, 25	7.9	7.0	11
April 27-28	25	1-25	6.3	8.3	—
May 27-28	25	1-25	7.0	7.6	—
June 23-24	25	1-25	4.6	4.4	4
July 14-15	25	1-25	5.3	6.0	—
August 9-11	25	1-25	7.0	2.4	66
August 30-31	22	1-22	6.8	3.2	53
September 22	13	1-13	5.0	3.2	36
September 27	12	14-25	5.0	4.2	16
October 10-11	20	(1-18), (19-25)‡	4.9	4.3	12
November 15-18	25	1-25	6.1	3.6	41
1899.					
January 4-6	7	15-17, 22-25	4.4	6.9	—
February 21-24	8	15-22	4.9	3.8	22
April 5-6	25	1-25	6.0	1.8	70
April 26-28	25	1-25	4.9	3.5	29
May 23-25	11	15-25	4.7	3.7	21
June 13-14	25	1-25	4.8	1.9	60
July 6-7	25	1-25	5.1	2.0	61
September 14	13	6-18	3.7	3.1	16
September 28	6	1-3, 6, 19, 20	3.6	4.1	—
December 16-19	25	1-25	4.9	5.6	—
1900.					
January 10	10	1-10	5.8	1.8	69
January 16	6	11-16	6.0	4.5	25
January 20	9	17-25	6.5	5.5	15
February 1	5	1-5	6.7	4.8	28
April 12-13	21	1-21	6.2	6.9	—
April 25	8	1-8	5.8	5.7	2

*In a few cases, where three days, only, elapsed between successive scrapings, the data for three days before and after scraping have been included in the table.

† A dash designates a loss rather than a gain.

TABLE No. 11—(*Continued*).

(1)	(2)	(3)	(4)	(5)	(6)
May 1	8	9–16	5.5	5.0	9
May 25	13	10–22	5.8	3.5	40
June 7–9	14	1–12, 22, 23	4.6	3.7	20
June 27	12	13–21, 23–25	4.0	3.3	18
July 6	12	1–12	4.0	4.4	—*
July 16	12	13–20, 22–25	5.0	4.0	20
July 27	12	1–12	3.8	2.6	32
August 7	9	12–14, 19–20, 22–25	4.1	4.2	—
August 16–17	12	1–12	3.7	3.5	5
August 28–30	8	12, 13, 17–20, 22, 23	3.8	4.1	—
September 5–6	14	1–9, 11, 16, 21, 24, 25	3.9	3.6	8
September 14	13	10–20, 22–23	4.0	3.2	20
September 18	12	1–9, 21, 24, 25	3.6	3.4	6
September 25–26	9	14–22	3.4	4.3	—
October 11, 12	5	7, 11–13, 23	4.0	3.4	15
October 18	5	1–5	4.3	4.0	7
October 23–25	11	14–22, 24, 25	4.9	4.2	14
November 13	5	3–4, 22–25	4.2	3.5	17
November 20–21	20	1, 2, 5–21, 23	4.9	3.8	22
December 7–8	22	1–21, 25	5.2	6.0	—

* A dash designates a loss rather than a gain.

caused an increased loss of head until it has escaped. Winter disturbances have at times produced such effects that scraping of the surface has not increased the quantity filtered; moreover, the yield of certain beds, which have recently been repaired, was not increased appreciably by scraping, prior to reconstruction.

Table No. 11 has been prepared from the filter records, in order to show the actual conditions of yield, expressed as average losses of head, during periods of 4 days directly before and after scraping. These data have been tabulated only for such times as when at least 5 beds were scraped and there was no interference in the conditions, due to cleaning, during the 4 days before and after. The number and designation of the beds scraped is entered with the dates. The actual average loss of head is given for the 4 days prior to cleaning, and that for the same period afterward has been obtained by adjusting the actual average to the rate of filtration and temperature of the water before scraping. For the years 1895 and 1896, the losses are not adjusted for rates, on account of insufficient data, but it has been assumed that these are the same before and after scraping, which supposition the results of other years have shown to be not seriously in error.

It will be noticed that the improvement in cold weather is less than at other seasons, and, at times, it appears that the loss of head was even greater after scraping than before. This condition was probably pro-

duced by entrained air caught in the filter when refilling, the effect of which is more evident with the low-temperature conditions of winter. Also, certain of the summer records occasionally show losses which cannot be accounted for in any other way than by the presence of air in the filter. It is known that the desire to return the filter to service quickly has often caused too rapid filling and subsequent starting of the pumps. During 1898 Mr. Walter E. Spear studied the effects of rapid refilling, and pointed out clearly, as shown in a diagram published in the report of the Lawrence Water Board for that year, that hastened refilling oftentimes lessened the yield of the filter for a few days.

The difficulty caused by entrained air can be remedied by back-filling from below, filtered water entering through the underdrains and passing upward through the sand to a proper elevation before turning on the supply of raw water; by which method the air will be forced out of the bed. Care must be observed, however, not to back-fill so rapidly as to disturb the sand about the underdrains, but this danger can be wholly avoided by providing a back-filling pipe capable of being so regulated that not more than the maximum permissible rate of refilling can take place under the existing pressure conditions.

It is also noticeable that, since the early operation of the filter, the greatest benefit from cleaning the surface was secured during the last two years; this undoubtedly was due to the more thorough and careful management under which the filter has recently been operated.

Intermittence of Operation.—It was intended, when the filter was built, that it should be operated intermittently, or, in other words, that there should be a daily period of rest, during which the water in the bed would be drawn out and the sand filled with air to the bottom. This condition was regarded as necessary to insure the "burning up" of the organic matter retained in the filter.

It was endeavored to give the filter this daily rest during its early history, but various circumstances, such as the demands of consumption and the securing of equally favorable results with continuous operation, has led, especially in recent years, to practically continuous filtration, except during times of scraping. From Table No. 4 the average number of hours per day during which the pumps were not operated may be determined; and, realizing that at least 3 hours should elapse between turning on the water and starting the pumps, it will be

seen that during recent years there has been but little opportunity to fill the sand with air. The record of drainings is given in Table No. 12.

TABLE No. 12.—Number of Times when the Lawrence Filter was Drained, from 1894 to 1900, Inclusive.

Months.	1894.	1895.	1896.	1897.	1898.	1899.	1900.
January	17	16	6	7	6	14	4
February	15	8	9	11	4	8	6
March	30	10	9	12	5	7	6
April	23	13	13	10	9	5	9
May	26	19	12	9	13	12	8
June	26	15	14	14	8	11	6
July	16	23	17	12	10	10	9
August	27	20	19	15	9	14	12
September	22	15	22	12	5	12	8
October	22	19	16	13	4	14	13
November	24	15	13	13	5	12	6
December	17	10	4	8	13	12	3
Total	265	183	154	136	91	131	90
Average per month.	22	15	13	11	8	11	8

Cost of Additional Construction, and Expense of Maintenance.

Additional Construction Items.—Since placing the filter in service, September 20th, 1893, there have been additional accounts which might properly be charged to construction. These expenditures include the purchase of machines to aid in the work of maintenance, the building of a new driveway by which the cost of conveying has been reduced, and the paving of the sloping bank, thus enlarging and improving the storage place for washed sand. The original cost of the filter, as given in the Report of the Water Board for the year 1893, was $65 459.67. The additional items, expressed in rounded figures, are given in Table No. 13.

TABLE No. 13.—Additional Construction.

1894.	Labor and material, as per Report of Water Board	$4 070
1895.	Cost of washer, including concrete work	540
	Labor account given in Annual Report, minus that which is properly chargeable to maintenance	2 700
1896.	Cost of ice elevator and conveyer	2 730
1899.	New driveway and paving of storage slope	3 600
1900.	" " " " "	1 240
		$14 880

These additional items, together with the original cost, make the total expenditure for construction about $80 000.

Cost of Maintenance.—One of the most interesting elements of a study of the Lawrence filter has been the annual cost of the several operations. Various figures appearing in early reports of the Water Board have been misleading, because they have included expenses of grading, care of grounds, etc., which, in accordance with the usual custom of charging such special and unclassified items to some main account, burdened the filter charges. This circumstance originated from the desire to have constantly at hand (for expeditious cleaning of the filter, etc.) an ample laboring force, which was employed upon odd jobs whenever there was not sufficient routine work.

The writers believe that the data of expense presented in the following tables are as nearly correct as can be obtained at this date, because of the length of time which has elapsed since the first record. In order to render the figures as reliable as possible, a study and a transcript of all important records were made, together with a comparison of the timekeeper's weekly report, with the daily timebooks, and finally both of these were compared with the filter record books, showing the extent and nature of the work of maintenance for each day. Notwithstanding the degree of care exercised in the following compilations, the insufficiency of some of the data has made it seem advisable to round up all figures of cost, a process which it is believed will also simplify comparison.

TABLE No. 14.—ITEMIZED STATEMENT OF COST OF MAINTENANCE OF THE LAWRENCE FILTER, FOR THE YEARS 1895-1900.

Items.	1895.	1896.	1897.	1898.	1899.	1900.
Scraping	$2 550	$1 350	$1 950	$1 930	$2 060	$2 130
Scraping and sanding		1 460	840	300	1 270	1 250
Conveying	1 320	1 180	2 200	1 660	1 470	1 410
Washing	1 000	2 120	1 950	1 600	1 450	1 510
Removal of snow and ice	2 780	2 350	2 330	3 480	2 300	2 330
General	450	490	850	710	500	720
Total	$8 100	$8 950	$10 120	$9 680	$9 050	$9 350
Per million gallons	7.80	8.10	9.00	8.00	7.70	7.70
Total, minus snow and ice	5 320	6 600	7 790	6 200	6 750	7 020
Per million gallons	5.10	6.00	6.90	5.10	5.80	5.80
Special repairs	.....	.....	.....	4 830	.....	2 260
Total pumpage for year in million gallons	1 040	1 100	1 130	1 210	1 170	1 210

NOTE.—The writers were unable to classify the cost of maintenance during the period of operation of the filter in 1893 and during 1894. The total expenditures for this purpose during 1893 (3½ months) amounted to $1 210, and in 1894 to $4 610.

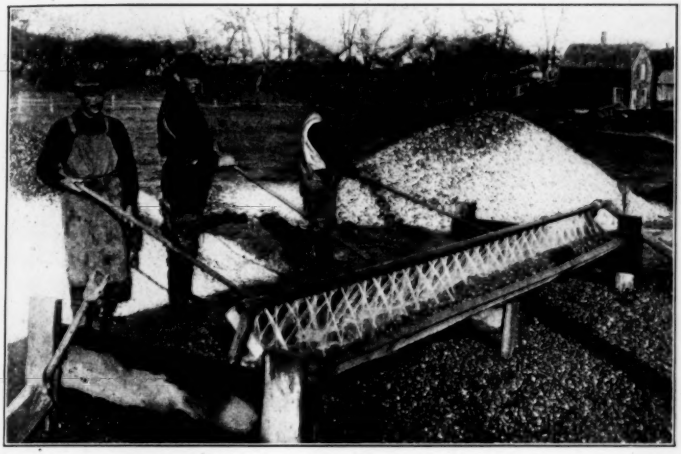


FIG. 1.—METHOD OF WASHING GRAVEL.

FIG. 2.—WHEELING OUT SAND, AFTER SCRAPING, LAWRENCE CITY FILTER.

Classification of Accounts.—In explanation of the items in Table No. 14, the following statements are made to show the nature and extent of the work done under each classification.

1.—Scraping, includes the removal of the surface layer of dirty sand by means of shovels and throwing into small piles, about 10 to 12 ft. apart, upon the surface of the bed; together with wheeling the sand from these heaps to a temporary dump along the lower driveway on the north side of the filter. This process also includes raking and smoothing the beds after cleaning.

2.—Sanding, includes removing the clean sand from the storage bank along the driveway to the surface of the bed being sanded, distributing it in layers, and smoothing the surface. The cost of sanding includes the expense of the scraping necessary to prepare the surface of the filter beds for this operation.

3.—Conveying, includes the loading of the dirty sand and its transportation in carts from the temporary dump on the lower driveway to a permanent dump close to the washer. During the early years, before the bank of clean sand was placed as advantageously as at present along the paved slope to the driveway, this item also included the loading and transportation from the dump of clean sand to temporary piles along the lower driveway, close to the beds on which it was to be used for sanding.

4.—Washing, includes the screening of the dirty sand at the first or traveling hopper of the washer and the transportation from the draining boxes to the permanent storage bank, together with such attention and repairs as the washer itself requires.

5.—Removal of snow and ice, includes, for snow, the loading and transportation from the frozen filter surface in carts or sleds to the river bank. For ice, the item includes cutting and moving cakes to the ice-conveyer, feeding the same and taking care of the delivery over the river bank, together with the cost of coal and such attention as the machinery demands. This last account also includes, during the early history of the filter, and, to a much less extent, at the present time, the loading and transportation of ice in carts or sleds from the filter surface.

6.—General, includes miscellaneous items not appearing in the foregoing classification, such as care of filter banks and lawn directly adjacent, repairing and cleaning main distribution channel and lateral

carriers, and repairs to driveway and paved bank. All repairs of tools and machinery are charged to the proper accounts of the work upon which they are used. The expenditures under this item are not constant, but fluctuate according to the amount of repair work necessary each year.

Detailed Study of Scraping.—It has been the custom, when it was the intention to scrape a portion of the filter upon a given day, to shut off the inlet water during the evening before, and, by running the pumps at gradually reduced speed, drain the sand so that men could work upon it the next morning. Since January, 1900, however, water has been pumped from the surface of the filter back to the river when draining. Thus, it has not been necessary to close the inlet gate as early as formerly, or to have so much of the work done in an uneconomical manner by the large pump in the station. But the effect, which is of greater importance, is that the filter surface can now be made ready for cleaning at a definite time, whereas, previously, because of the uncertainty of draining through a more or less clogged material, the filter was frequently not in a condition for work when desired.

Laborers first scrape the surface, removing the *schmutzdecke*, varying from $\frac{1}{2}$ to 1 in. in thickness. Sandals are not worn on this work, as is customary at some other plants. When a few beds only are cleaned, this part of the work is usually entirely completed before removing any of the sand; but if a large area is being scraped, the two operations are often performed alternately. Until recently the usual shovels with short handles and square mouths have been used, but at the present time special long-handled shovels of extra width have been tried with success and economy.

The wheeling-out is done in barrows upon planks laid upon the sand surface, and the length of carry varies from 75 to 150 ft. This work is shown in Fig. 2, Plate IX. The surface of the bed compacted by the process of cleaning is then made porous or loose by the use of rakes with teeth about 2 ins. long, and, finally, the raw surface is evened with a thin smoother attached to a handle and used like a rake. For some time between 1895 and 1899 special rakes, with teeth about 6 ins. long, were used to loosen the upper portion of the sand after scraping. That this process permitted the water to pass through the sand more freely is probably true, but it undoubtedly tended also to produce each time a

deeper clogging in the sand. It is therefore believed that it was a prudent measure to abandon their use in favor of the short-toothed rakes, which are entirely sufficient to loosen the surface of the sand compacted by the laborers in working without sandals during scraping.

Unfortunately for the purpose of comparison, the cost of scraping has not always borne an exact relation to the amount of work performed. This has been largely due to the uneconomical methods used, some of which will necessarily obtain until greater expenditures for construction and equipment are allowed. Whenever a few beds only are scraped, the work is performed by the usual maintenance force always at the plant, varying in number from 8 to 15 men. At times, when the demand for water renders it necessary to scrape a large part of the whole surface at once, additional laborers are drafted from the street and pipe repair force. As many as 30 men have at times been used, many of whom were unfamiliar with the best methods of performing this kind of work. This extravagant method of cleaning has been used somewhat frequently, and is a result of the necessity of scraping as large a unit area as possible at one time. A few years since, one of the writers of this paper prepared a statement of the amount of work accomplished with varying numbers of men employed, showing quite conclusively that 10 men was the most economical number to use during scraping as then carried on, because larger numbers caused some to be in the way of others.

Table No. 15 presents a summarized statement of the cost of scraping in comparison with the amount of work done.

TABLE No. 15.—Statement of the Total Number of Beds Scraped per Year, together with the Cost per Bed per Scraping, and per Million Gallons Filtered.

Years.	Number of bed scrapings.	Cost of scraping.	Cost per bed.	Cost per million gallons of water filtered.
1896	304	$1 350	$4.40	$1.20
1897	416	1 950	4.70	1.70
1898	331	1 930	5.80	1.60
1899	421	2 060	4.90	1.80
1900	395	2 130	5.40	1.80

Some recent studies show that the costs in Table No. 15 are divided among the several items about as follows: Wheeling out, $\frac{1}{3}$; scraping,

$\frac{1}{3}$; and smoothing of surface, $\frac{1}{6}$ of the total. These figures show at once in what portion of the work a saving should and can be made, namely, in the use of some approved form of conveyer to replace the wheel-barrow. The scraping of single beds has occasionally cost somewhat less than $3, although in cold weather, with a frozen sand surface, the cost has been as high as $10 per bed.

Detailed Study of Sanding.—This process requires but a short description. In the early history of operations, all the conveying charged to this item was done with wheel-barrows; but, recently, it has been the custom to place, over the bed to be sanded, a platform, level with the lower driveway, upon which the teams deliver the clean sand and from which it is wheeled to position by barrows. The method of sanding is about as follows: The bed is scraped as usual, the upper 10 ins. is removed and placed at one side, then such an amount of clean sand is used in the excavated trench as will bring the bed to grade when the old sand is replaced upon it, and the surface shaped and smoothed.

It is probable that but little sanding was done in 1895, and, in consequence, the amount of work required in 1896 was considerably increased. For this reason, and also because less economical methods were used than later, the cost of sanding during this year was unusually high. The reduced cost in 1897 may be explained in part by the fact that a less depth of sand was added during this year, and also, perhaps, because certain expenditures classed under "Conveying" should rightfully be charged to this work. The figures in 1898 include simply the cost of sanding Beds Nos. 1, 2, 3 and 12.

It appears from the records of the years 1899 and 1900 that the average cost of sanding a bed is about $50. It has been found that on the average the number of scrapings per bed per year (1896–1900) has been about 15, and the average cost of each about $5.00, making $75 approximately the total annual cost of keeping a bed clean. From this figure the economy of bringing the beds to grade by sanding once a year is readily apparent.

Detailed Study of Conveying.—The statistics for conveying are less satisfactory than any others in connection with the filter, for two reasons: First, the methods have not been economical, although much improved recently; secondly, the account is less accurate than others because of the difficulty which has arisen in special instances

in determining whether certain expenditures belong to this item or to others, like sanding or general maintenance work, and at times to hauling-accounts entirely foreign to the work of the filter. The account for 1897 is probably higher than it should be, but the writers can find no authority for changing from the amount given.

The rough contour of the ground north of the filter renders it necessary to use carts in hauling the dirty sand to the dump near the washer; whereas, if the area were more nearly level, cars upon a tramway could be operated by the laborers or by some smaller force than present conditions require. The extravagance of the current methods has been realized, and the trial of a machine-conveyer or some means of hydraulic transportation has been recommended, but the money for such installation has not been made available.

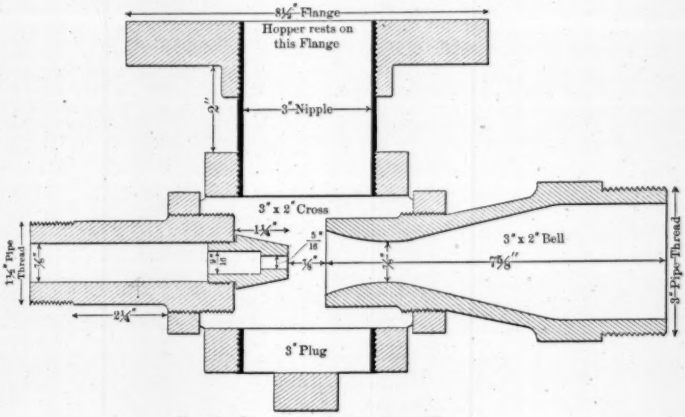


FIG. 3.—DETAIL OF JET OF SAND WASHER.

Detailed Study of Washing.—The washing machine used at Lawrence is, so far as the writers are aware, the only one of its kind in this country, and is similar to the design of some washers in service in Europe. The bowls of the machine are four in number and are light sweep-castings made at a local foundry. The sand drops to the bottom of each bowl or hopper and is caught by an inflowing horizontal jet and carried across and upward through a 3-in. pipe into the next bowl. The fitting at the bottom of the hopper is a standard 3-in. cross, with a plug in the bottom for the purpose of cleaning. Into a bushing in the inlet end of

the cross is placed a $1\frac{1}{2}$-in. pipe, to the end of which is attached a removable chilled-steel nozzle made from a piece of hexagonal rod. The nozzles in use have $\frac{5}{16}$-in jets, although other sizes have been tried. At the outlet of the cross there is screwed a special 3 x 2-in. reducer, the small end of which is placed within the cross and is of bell shape in order to readily receive the mingled water and sand. These special fittings, which wear out continually, are made as required at the shop of the Water Department. A sketch of the special nozzle, with all the attached fittings, is shown on Fig. 3.

The washer was built and erected early in 1895, and was first used on June 17th. The cost of the patterns, castings, piping, lumber, and labor on the machine was about $150, and that of the concrete pavements and drains about $240, while the labor account of the department for setting up, etc., was about $150, making a total cost of $540. Additional construction, new traveling hopper, draining boxes, etc., have been charged to the yearly maintenance accounts.

The presence in the sand of small stones, which would occasionally drop into the jet and stop the flow of the washer, rendered it necessary to screen all sand, both the old and dirty as well as the new, before throwing it into the first hopper. It was formerly the practice to screen the sand at the storage bank and wheel it to the washer, a process which required two men, and caused an irregular and intermittent feeding. During the summer of 1898, one of the writers experimented with different arrangements for transporting the sand, and, in a short time, devised the following method, which was adopted: A traveling hopper, mounted upon a light frame of piping and placed at will wherever dirty sand is dumped, is fed from a 2-in. pipe from the main supply. Sand thrown into the hopper is carried by the entering water into and through a connecting length of 3-in. pipe to the main washer, where it is raised about 5 ft. into the first hopper of the machine. The nozzle of the traveling hopper is $\frac{1}{2}$ in. in diameter. The 3-in. pipe line varies in length from time to time, and is made of running lengths of pipe connected in such lengths as are necessary for the distance. This method has been in use since July, 1898, and has never failed in carrying the sand to the washer from the most distant point required, about 200 ft. being the maximum tried up to the present time. It appears that the sand has not only been washed more thoroughly since using this apparatus, due probably to

a more complete mixing and scouring in the pipe line, but, also, there has been no apparent increase in the amount of water used, and, moreover, there has been a material reduction in the cost per yard.

From the last hopper, the cleansed sand is conveyed by water through a wooden trough to one of two draining boxes. Formerly, these boxes, each holding 3 cu. yds., were stationary, and after draining, the sand was shoveled into wheel-barrows and delivered along the dump for clean sand. In 1900 a further saving of time and labor was effected by the adoption of smaller boxes, or cars, on trucks, into which the sand is delivered from the washer, drained, and then carried on a narrow-gauge track to the storage bank where it is quickly dumped by removing one side of the car. The cost of these cars and track appears in the account for 1900. The washer, complete, with the old-style draining boxes, is shown in Fig. 1, Plate X.

Originally, the washing operations required 6 men, and more at times when the wheeling distance was considerable; now, 4 men do the entire work of screening, of attendance upon the machine, and of carrying away the clean sand to the dump. Thus, since the summer of 1898, the expense has been considerably less than before. Washing operations are generally begun each year about April 1st, and are suspended about December 1st. The ordinary water pressure at the machine, with everything running, is 63 lbs., and at the hopper 56 lbs. A statement of the cost of washing, the amount of sand washed, and the quantity of water used, with the unit factors for each, covering the last four years, and with less complete data for the years 1895 and 1896, is given in Table No. 16.

TABLE No. 16.—Details of Sand-Washing Operations at Lawrence Filter, for the Years 1895–1900.

Year.	Cost of Operations.			Cubic yards washed.	Cost per cubic yard.	Estimated number of cubic feet of water used.	Approx. Volume of water to sand
	Labor.	Bills.	Total.*				
1895	$990	$10	$1 000	1 500	$0.67	.....	.....
1896	2 110	10	2 120	.....	.....	.....	.....
1897	1 920	30	1 950	2 900	0.67	1 050 000	14
1898	1 540	60	1 600	2 600	0.62	970 000	14
1899	1 340	110	1 450	3 200	0.45	1 140 000	13
1900	1 290	220	1 510	3 000	0.50	820 000	10

* Cost of wash-water is not included.

The normal maximum amount of sand which can be washed in one day is 21 cu. yds., for which the item of labor of 4 men at $2 per day represents a cost of 38 cents per yard. The charges over and above this amount represent bills and labor for repairs and additional parts of the machine, necessary each year when the washer is made ready for service, and the cost of delays due to stopping the machine, shifting positions, and any other causes which tend to decrease the output below the maximum.

Detailed Study of Removal of Snow and Ice.—During the winter weather, that is, in the months of January, February and March, scraping can seldom be accomplished without the necessity of removing ice from the surface of the filter. Whenever the ice over that portion which is to be scraped is covered by snow, the latter is shoveled into carts and taken away. It was formerly the custom, also, to remove the ice in the same way after cutting; but, recently, this has been done at times only when teams are at hand, and more ice must be removed than the ice machine will handle. A layer of ice, about 15 ins. thick, in direct contact with the drained sand surface, is shown in Fig. 2, Plate X.

An ice-conveying machine was procured and set up during the winter of 1895–96, and since January 18th, 1896, has been in almost constant use during the winter season. The machine, quite compact and convenient, combines the power and the runway of the customary ice-house conveyer on a single framework; the whole being supported upon a car and truck moving upon a wide-gauge track. The boiler, engine and machinery is housed on the body of the car, while at one end is supported the framework of the elevator. This elevator is the usual style of long-link conveyer running over a power drum at the top and an idler at the bottom, and is supported upon a frame between guides. The runway of the conveyer can be placed at any slope, but when in use is at an angle of about 45° with the horizontal. The engine will develop 11 H.-P., and uses about 350 lbs. of coal per day when running. The conveyer travels at a rate of from 70 to 100 ft. per minute, and will hold 6 cakes of ice at once. The machine is capable of propelling itself along a track on the embankment, so that any part of the south side of the filter can be reached. The ice must be cut into cakes of a suitable size and fed to the conveyer, which then delivers it over the embankment upon a runway into the river, or

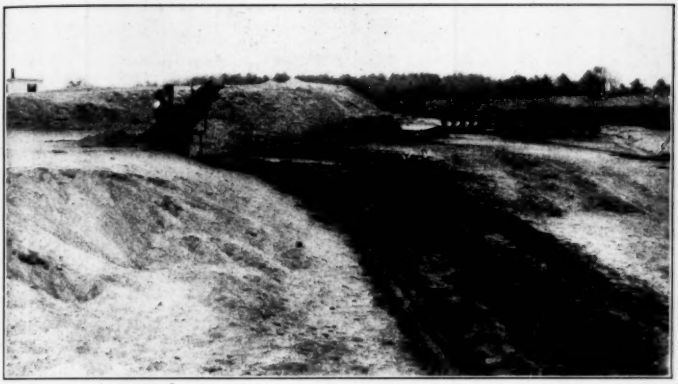


FIG. 1.—WASHING MACHINE AND TRAVELING HOPPER, LAWRENCE CITY FILTER.

FIG. 2.—THICK ICE ON SAND SURFACE AT TIME OF DRAINING LAWRENCE CITY FILTER.

on its frozen surface. If the ice is sufficiently thick, it is marked with a horse plow, and long cakes are then broken off and divided into small pieces capable of being caught by the carrier bars of the elevator. Inasmuch as a considerable amount of labor is required to cut the ice into small pieces, a saving might be effected with a more powerful engine, capable of moving a wider elevator at greater speed.

Ice has quite often appeared upon the filter surface by the middle of November, but becomes hardly thick enough to cut for several weeks, and generally disappears by the middle or last of March. The removal of ice over such beds as require cleaning is accomplished just before it is desired to scrape. If the cleaning of an adjoining area occurs shortly afterward, and the formation of ice does not prevent, it frequently happens that the ice already formed on the beds in question may be floated over the area scraped and its removal consequently avoided until some future date. The ice is rarely allowed to become thicker than 15 ins., and, generally, three or four crops are cut over portions of the surface.

The cost of removing snow and ice, in order to clean the filter in the winter time, is an expensive operation, and has varied from $1.90 to $2.90 per million gallons, according to the severity of the winter and the methods used. The average cost of removal has been $2.30 per million gallons filtered in a year, or about $1 100 per year per acre of net filter surface. The demands of consumption are such that the filter must be scraped as usual in the winter time, and the cost of ice removal is, therefore, greater than at other places where reserve filtering area is provided. Other than conveying, this item is probably the most expensive and wasteful of any part of the maintenance. The average yearly cost per acre, capitalized at 4%, is $27 500, a sum much in excess of the cost per acre necessary to erect a cover over the present filter, and greater, even, than the additional cost per acre of an approved form of covered masonry filter over a similar one not so covered.

The Efficiency of the Filter as Determined by Chemical and Bacteriological Analyses, and by the Decreased Prevalence of Typhoid Fever in the City of Lawrence.

General.—From the time of first operating the filter, the results of its work have been studied by the Massachusetts State Board of Health, which has conducted chemical and bacterial examinations of

the water of the Merrimac River as it flows upon the filter, and of the effluent of the filter at the pumping station, and after passing successively through the distributing reservoir, through the supply mains to City Hall, and, at length, at the extreme easterly end of the city to the Lawrence Experiment Station. It has been customary, especially during later years, to make daily bacterial analyses of the water at the several points just mentioned, during the months from November to April inclusive, and weekly examinations during the warmer season from May to October, when it has been found that the filter is more uniformly in a state of biological stability. The results of these studies have been published in detail in the Annual Reports of the State Board of Health, from which nearly all the data presented in the subsequent tables have been derived.

The water supply of Lawrence is for the most part pumped after filtration to an open distributing reservoir located upon a hill about $\frac{3}{4}$ mile from the river.* The reservoir consists of two basins of about equal size, having a combined capacity of 40 000 000 galls., equivalent to nearly two weeks' supply for the city. The inlet is arranged to partially aerate the water entering the reservoir by discharging into the air and allowing the water to fall upon a series of masonry steps over which it flows into both basins. From the reservoir the water travels through main distribution pipes for a distance of about $1\frac{1}{2}$ miles to City Hall, and for a total distance of nearly $2\frac{1}{2}$ miles to the Experiment Station.

Chemical Results.—During the construction of the filter, ground-water was encountered at various points in the excavation, particularly in the central and easterly portions. The superficial area of the southerly slope of the hill leading from the filter toward the reservoir, apparently tributary to the filter, is perhaps 23 acres, and the above-mentioned flow of ground-water probably represents the yield from this or a more extended water-shed. In this territory there are at the present time about a dozen dwelling-houses, many of which have by this time been connected with the sewer system, but some are yet provided with individual cesspools or privies.

The average yearly results of the weekly chemical examination of the water of the Merrimac River, and of the effluent from the filter

* Since 1896, about one-tenth of the water has been pumped independently to a closed stand-pipe just above the reservoir, to supply the people living upon the two main hills.

TABLE No. 17.—Results of Chemical Analyses of Applied Merrimac River Water and Effluent from Lawrence Filter for the Years 1893–1899 (Parts per 1 000 000).

Year.	Yearly rainfall, in inches.	Color.			Solids.						Chlorine.			Hardness.		
					Total.			Loss on ignition.								
		River.	Effluent.	Percentage removed.	River.	Effluent.	Percentage removed.	River.	Effluent.	Percentage removed.	River.	Effluent.	Percentage removed.	River.	Effluent.	Percentage removed.
1893, 3 months	.....	0.39	0.33	15.4	44.3	.....	.....	.....	.....	.....	2.1	2.2	—4.8	.....	.....	.....
1894.....	31.35	0.42	0.34	19.0	37.0	61.0	—64.9	13.0	14.1	—8.5	2.1	2.5	—19.0	16	31	—94
1895.....	39.05	0.46	0.39	15.2	43.4	59.5	—37.1	17.5	17.0	2.9	2.4	2.8	—16.7	17	24	—41
1896.....	36.68	0.44	0.36	18.2	39.8	54.3	—36.4	15.2	16.4	—7.9	2.0	2.2	—10.0	13	19	—46
1897.....	41.77	0.45	0.42	6.7	38.4	51.7	—34.6	15.4	16.8	—9.1	2.1	2.3	—9.5	13	18	—38
1898.....	49.43	0.45	0.41	8.9	39.0	46.6	—19.5	16.3	16.9	—3.7	2.1	2.3	—9.5	12	17	—42
1899.....	34.87	0.36	0.33	8.3	39.9	44.4	—11.3	14.9	14.2	4.7	2.5	2.6	—4.0	12	16	—33
Average, six years	38.86	0.43	0.38	11.6	39.6	52.9	—33.6	15.4	15.9	—3.2	2.2	2.4	—9.1	14	21	—50

Year.	Dissolved oxygen in river, percentage of saturation.	Nitrogen as												Oxygen Consumed.		
		Free ammonia.			Albuminoid ammonia.			Nitrites.			Nitrates.					
		River.	Effluent.	Percentage removed.	River.	Effluent.	Percentage removed.	River.	Effluent.	Percentage removed.	River.	Effluent.	Percentage removed.	River.	Effluent.	Percentage removed.
1893, 3 months.	92	0.089	0.086	3.4	0.200	0.110	45.0	0.004	0.005	—25	0.14	0.37	—160	4.2	2.9	31
1894.....	81	0.083	0.083	0.0	0.194	0.095	51.0	0.001	0.001	0	0.12	0.39	—220	3.9	2.6	33
1895.....	.....	0.097	0.095	2.1	0.202	0.097	52.0	0.002	0.001	50	0.14	0.38	—170	4.4	3.0	32
1896.....	.....	0.094	0.084	10.6	0.198	0.096	51.5	0.001	0.000	100	0.14	0.36	—160	4.2	2.7	36
1897.....	.....	0.082	0.107	—30.5	0.203	0.098	51.7	0.001	0.000	100	0.14	0.43	—210	4.5	3.0	33
1898.....	.....	0.065	0.099	—52.3	0.193	0.097	49.7	0.001	0.000	100	0.13	0.45	—250	4.4	2.9	34
1899.....	.....	0.113	0.083	26.5	8.203	0.083	59.1	0.003	0.001	68	0.15	0.36	—140	3.6	2.3	36
Average, six years	.....	0.089	0.092	—3.4	0.199	0.094	52.8	0.002	0.001	50	0.14	0.40	—190	4.2	2.8	33

Note.—The annual rainfall given in the table is the total annual precipitation at Lawrence, observed by the Essex Company on behalf of the United States Weather Bureau.

The "Solids" represent the average monthly determinations at the Boston Laboratory of the Massachusetts State Board of Health.

The determinations of color, solids and oxygen consumed on river samples were made after passing samples through filter paper. The color standards used were natural waters, the color of which had been accurately determined by comparison with Nesslerized ammonia standards, prepared as described on page 531, "Special Report on Examination of Water Supplies," and on page 329 of Annual Report of 1892, Massachusetts State Board of Health.

from its first operation to the end of the year 1899, are presented in Table No. 17.

Bacterial Results.—Table No. 18 shows, by months, the average numbers of bacteria in the water of the Merrimac River, as applied to the filter, and in the effluent of the filter at the pumping station and in the tap water at City Hall. During 1897, regular examination of samples collected at these points for *Bacillus Coli communis*, the representative bacterium indicating sewage contamination, was begun, and these studies have been continued to the present time. Where information regarding the occurrence of this organism has been available it has been included in the table.

The decreased efficiency of the filter during cold winter weather, especially in the first years of operation, has undoubtedly, in part, been due to the fact that the removal of ice is attended with such difficulty that only a portion of the filter can be scraped at any given time; while over the remainder of the area, when the filter is drained, the ice settles, becomes frozen to, and thus disturbs, the upper sand layers. On account of the many inherent difficulties attending the scraping of the filter in this weather, and because the cleaning of the beds nearest the pumping station was more effective in producing an increased flow, it oftentimes happened that only this portion of the filter was scraped. It is evident that the practice of scraping a part only of the surface at any given time, tends to produce high rates of filtration through those portions having a clean surface, and permits correspondingly lower rates through the area over which a more or less impervious *schmutzdecke* has already formed. Moreover, the drawing down of the water in the sand below the surface, the filter being uncovered, allows the upper layers of sand to become partially frozen, the effect of which is unquestionably disastrous to securing the highest bacterial efficiency.

Results of examination of samples of water from the outlet of the open distributing reservoir, although not included in the table, are, nevertheless, interesting because they show a considerable increase in the number of bacteria over the numbers in the effluent immediately after passing through the filter. It is probable, therefore, that the large increase in the numbers of bacteria found in the tap water at City Hall, principally during the warmer seasons of the year, as shown by the table, is either due to the organisms entering the open reservoir from

TABLE No. 18.—NUMBER OF BACTERIA IN THE WATER OF MERRIMAC RIVER, IN EFFLUENT OF FILTER AT PUMPING STATION, AND IN TAP WATER AT CITY HALL.

Month.	Number of days having complete series of samples.	Average Number of Bacteria per Cubic Centimeter.		Percentage of reduction in passing through filter.	Average number of bacteria per cubic centimeter in tap water at City Hall.	Further percentage of reduction in passing through reservoir and pipes.
		In water applied to filter.	In effluent of filter at pumping station.‡			
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1893.						
October†....	24	21 000	560	97.3	130	77
November*..	15	7 700	190	97.5	71	64
December....	12	9 400	200	97.9	100	50
Average.....	.....	12 700	320	97.5	100	69
1894.						
January.....	23	7 600	140	98.2	56	60
February....	21	8 300	250	97.0	89	64
March.....	20	6 900	220	96.8	110	50
April.....	23	14 800	230	98.4	82	64
May.....	‡	6 000	130	97.8	130	0
June.....	‡	8 300	110	98.7	95	14
July.....	‡	2 400	25	99.0	92	—268
August.....	‡	3 100	36	98.8	140	—289
September..	‡	6 500	42	99.4	85	—102
October.....	‡	25 300	120	99.5	81	32
November...	‡	16 600	180	98.9	72	60
December....	22	23 800	360	98.5	270	25
Average.....	.....	10 800	150	98.6	110	27
1895.						
January.....	27	18 700	210	98.9	140	33
February....	24	15 000	280	98.1	100	64
March.....	26	20 800	400	98.1	100	75
April.....	15	8 400	84	99.0	66	21
May.....	‡	7 000	68	99.0	170	—150
June.....	‡	9 000	68	99.2	64	6
July.....	‡	10 000	50	99.5	56	— 12
August.....	‡	5 000	38	99.2	55	— 45
September...	‡	5 000	40	99.2	50	— 25
October.....	‡	19 000	60	99.7	51	15
November...	17	8 700	64	99.3	50	22
December...	24	6 700	84	98.7	60	29
Average.....	.....	11 100	120	98.9	80	33
1896.						
January.....	27	7 500	170	97.7	88	48
February....	22	12 600	320	97.5	170	47
March.....	17	6 100	140	97.7	98	30
April.....	25	3 800	40	98.9	57	— 42
May.....	‡	4 600	64	98.6	50	22
June.....	‡	10 000	58	99.4	71	— 22
July.....	‡	3 400	34	99.0	120	—247
August.....	‡	3 700	290§	92.2	120	60
September...	‡	13 800	35	99.8	100	—197
October.....	‡	9 900	31	99.7	44	— 42
November...	‡	5 800	50	99.1	39	24
December....	24	9 700	67	99.3	58	13
Average.....	.....	7 600	110	98.6	85	23

See notes on page 302.

TABLE No. 18—(*Continued*).

(1)	(2)	(3)	(4)	(5)	(6)	(7)
1897.						
January	22	13 300	91	99.3	49	46
February	23	13 100	79	99.4	48	39
March	26	12 300	67	99.5	62	8
April	25	6 900	47	99.3	51	— 8
May	‡	4 500	35	99.2	73	—108
June	‡	4 000	40	99.0	100	—150
July	‡	6 300	110	98.3	300	—173
August	‡	10 200	70	99.3	100	— 43
September	‡	34 900	68	99.8	74	— 9
October	‡	13 400	31	99.8	78	—152
November	25	6 600	27	99.6	43	— 59
December	26	5 600	24	99.6	26	— 8
Average		10 900	57	99.5	84	— 47

Month.	Number of days having complete series of samples.	Average Number of Bacteria per Cubic Centimeter. Total in river water applied to filter.	*B. Coli communis.*	Total in effluent of filter at pumping station.‖	Occurrence of *B. Coli communis* in effluent at pumping station.	Percentage reduction in passing through filter.	Total number of bacteria per cubic centimeter in tap water at City Hall.	Occurrence of *B. Coli communis* in tap water at City Hall.	Further percentage reduction in passing through reservoir and pipes.
1898.									
January	26	6 500	....	39	.....	99.4	34	.....	13
February	17	4 700	....	45	.....	99.0	28	.....	38
March	25	3 700	....	34	.....	99.1	62	.....	— 82
April	23	2 300	....	21	.....	99.1	30	.....	— 43
May	‡	2 600	....	47	.....	97.6	19	.....	60
June	‡	6 400	....	47	.....	99.3	39	.....	17
July	‡	2 900	....	18	.....	99.4	53	.....	—194
August	‡	3 800	....	32	.....	99.2	32	.....	0
September	‡	3 700	....	68	.....	98.8	54	.....	21
October	‡	4 200	....	26	.....	99.4	47	.....	— 81
November.	19	4 700	28	110	1(5 times)	97.7	150	1(4 times)	— 36
"	16	4 300	30	53	1(3 times)	98.8	140	1(3 times)	—164
December.	20	5 100	25	200	1(13 times)	94.3	260	1(11 times)	10
"	19	5 100	26	280	1(12 times)	94.5	260	1(11 times)	7
Average.	.....	4 400	....	65	.....	98.5	66	.....	— 2
1899.									
January	26	4 900	28	83	1(14 times)	98.3	90	1(5 times)	8
February	21	5 900	31	110	1(13 ")	98.2	69	1(5 ")	37
March	26	6 300	19	45	1(2 ")	99.3	42	1(3 ")	7
April	24	2 900	12	40	1(4 ")	98.6	33	0	18
May	14	1 900	21	17	0	99.1	17	0	0
June	‡	4 500	55	100	0	97.8	47	0	53
July	‡	3 000	63	14	1(once)	99.5	36	0	—157
August	‡	5 100	21	20	0	99.6	47	0	—135
September	‡	5 500	40	32	0	99.4	24	0	25
October	‡	9 700	25	100	0	99.0	24	1(once)	76
November.	25	9 800	60	46	1(6 times)	99.5	38	0	17
December.	25	9 800	140	56	1(5 ")	99.4	37	0	34
Average	.....	5 800	43	55‡	.....	99.1	42	.....	24

* Excluding results of November 27th, when suction pipe was lowered in pump-well.

† Filter put in operation September 20th, 1893. The average results of 11 days after starting were: River, 37 000; Effluent, 6 200; Percentage reduction in filter, 83.2.

‡ Samples collected from rising main from pump. Examinations made weekly.

§ Increase due to test on pumps from August 10th to 22d, and emptying of force main on latter date.

‖ Excluding records for those days when the filter underdrains were being repaired; also for the two succeeding days.

the air, or to those developed in the water under the favorable circumstances. Notwithstanding this, however, it will be seen that throughout the greater portion of the year, a material decrease in the number of bacteria in the tap water is effected during its passage from the pumping station.

Relation of Scraping to Bacterial Efficiency.—Table No. 19 presents all the available data relative to the numbers of bacteria found in the effluent of the filter during periods of 3 days before and 3 days after the scraping of different numbers of beds, at times when daily analyses of the water were made. The table shows the efficiency of filtration at these times.

Of the 40 scrapings tabulated, 3 indicate no change in efficiency due to cleaning; on 21 occasions there was a decrease, and on 16 an apparent increase in the percentage removal of bacteria. It is significant, however, that of the 17 times when 12 or more beds were scraped (*i. e.*, about one-half or more of the entire filter surface) there were only 4 occasions on which an increased percentage of removal resulted, and 3 times when there was no apparent change, the remaining 10 times causing a decreased efficiency ranging from 0.1 to 2.9 per cent. During December, January and February, there were 13 times when less than 12 beds were scraped, and 4 times when more than one-half of the entire filter surface was cleaned. Of this later number, two indicated an increase and two a decrease in efficiency.

Inasmuch as daily bacterial examinations of the water of the river and of the effluent have not been carried on during the warmer months of the year, from May to October, when the filter is presumably in its most stable condition, the table probably represents the average and the worst effects only, produced by scraping.

Relation of Continuous and Intermittent Operation to Bacterial Efficiency.—Statements have already been made to show that the Lawrenc filter has been operated in the past, for varying intervals of time, bot intermittently and continuously. It has been possible to collect and present in Table No. 20 for purposes of comparison, data representing, from a bacteriological standpoint, the results of these two methods of operation. Twelve of the 16 periods of continuous operation occurred immediately after cleaning some part of the filter surface, while during 8 of the 9 periods of intermittent action, scraping was carried on. Under these circumstances, it is difficult to draw definite conclu-

TABLE No. 19.—Average Numbers of Bacteria in Effluent of Lawrence City Filter during Periods of Three Days Before and After Scraping, and Percentage Relation to Average Number in Applied River Water.

Date of scraping.	Number of beds scraped.	Designation of beds scraped, by numbers.	Average Number of Bacteria per Cubic Centimeter in Effluent during Three Days.		Average Percentage Removal from Applied River Water during Three Days.		
			Before scraping.	After scraping.	Before scraping.	After scraping.	Percentage decrease.
November 21 1895	2	Nos. 24-25.	75	120	99.3	99.0	0.3
February 22. 1896	2	Nos. 11-12.	400	280	96.9	97.8	—0.9
April 23-24, 1896	3	Nos. 1-3.	26	28	99.0	99.1	—0.1
October 21, 1893	4	Nos. 20-23.	170	110	97.9	96.9	1.0
March 15, 1898	4	Nos. 22-25.	37	37	99.5	99.2	0.3
January 31, 1896	4	Nos. 1-4.	46	84	98.9	98.3	0.6
March 10, 1898	5	Nos. 10-14.	73	28	97.0	99.5	—2.5
March 10, 1896	6	Nos. 16-21.	97	110	98.1	98.8	—0.7
April 7, 1897	6	Nos. 15-20.	44	42	99.2	99.6	—0.4
December 1-2, 1897	6	Nos. 10-15.	34	30	99.2	99.3	—0.1
January 5-7, 1898	6	Nos. 9-14.	120	23	98.5	99.7	—1.2
February 16-19, 1898	6	Nos. 8-13.	51	57	98.7	97.7	1.0
Average first group	4.5		98	79	98.4	98.8	—0.4
February 12, 1898	7	Nos. 1-7.	31	51	99.3	98.7	0.6
January 4-6. 1899	7	Nos. 15-17, 22-25.	100	130	98.1	97.9	0.2
November 15, 1899	7	Nos. 1, 2, 11, 12, 22-24.	30	20	99.5	99.8	—0.3
Dec. 31. 1896-Jan. 1, 1897	8	Nos. 3-10.	88	120	99.4	98.9	0.5
February 21-24. 1899	8	Nos. 15-22.	110	140	96.7	97.6	—0.9
December 10, 1896	9	Nos. 1-9.	40	27	99.3	98.8	0.5
November 17-18, 1897	9	Nos. 1-9.	16	20	99.8	99.7	0.1
November 27, 1897	9	Nos. 1-9.	19	34	99.7	99.2	0.5
February 26, 1898	9	Nos. 1-9.	84	16	98.0	99.5	—1.5
Average second group	8.1		58	62	99.1	98.9	0.2
January 5-6, 1897	11	Nos. 1, 11-20.	120	110	98.9	99.6	—0.7
January 11-15, 1898	11	Nos. 15-25.	23	11	99.7	99.9	—0.2
April 13-14, 1897	12	Nos. 1-7, 21-25.	33	46	99.6	99.2	0.4
April 28-29, 1897	14	Nos. 1 -14.	64	32	99.0	99.0	0.0
November 10-11, 1897	16	Nos. 10-25.	54	13	99.2	99.9	—0.7
November 22-23, 1897	16	Nos. 10-25.	20	19	99.7	99.7	0.0
April 20-22, 1897	18	Nos. 8-25.	46	50	99.3	98.0	1.3
Average third group	14.0		51	40	99.3	99.6	—0.3
November 26-30, 1895	23	Nos. 1-23.	95	48	99.0	99.0	0.0
March 22-26, 1898	23	Nos. 1-22, 25.	15	21	99.5	99.0	0.5
April 26-28, 1899	24½	Nos. 1-4, 6-25, ½.	18	41	99.3	98.7	0.6
December 27-31, 1895	25	Nos. 1-25.	120	75	99.0	98.8	0.2
April 8-10, 1896	25	Nos. 1-25.	54	39	98.2	98.8	—0.6
March 23-31, 1897	25	Nos. 1-25.	42	34	99.7	99.6	0.1
December 11-14, 1897	25	Nos. 1-25.	18	20	99.7	99.8	—0.1
April 27-28, 1898	25	Nos. 1-25.	27	28	99.4	98.7	0.7
April 5-6, 1899	25	Nos. 1-25.	12	96	99.2	96.3	2.9
November 21-24, 1899	25	Nos. 1-25.	36	210	99.5	97.9	1.6
December 5-8, 1899	25	Nos. 1-25.	150	26	98.0	99.7	—1.7
December 16-19, 1899	25	Nos. 1-25.	34	53	99.8	99.3	0.5
Average fourth group	24.6		52	58	99.2	99.0	0.2
General average	13.0		67	62	99.0	99.1	—0.1

sions, but as far as the data represent actual conditions, there seems to be no practical difference in the bacterial efficiency obtained with the two methods of operation.

TABLE No. 20.—Comparison of Numbers of Bacteria in Applied River Water and Effluent of Filter, during Periods of Continuous and Intermittent Operation.

Periods (Inclusive).	Number of Days.		Average Number of Bacteria per Cubic Centimeter in		Percentage of bacteria removed by filtration.
	In period.	When filter was drained.	Applied river water.	Effluent of filter.	
With Continuous Operation.					
Dec. 14-27, 1897	14	0	5 700	26	99.6
Jan. 15-24, 1898	10	"	7 600	25	99.7
Jan. 26-Feb. 9, 1898	15	"	5 100	36	99.3
Feb. 9-24, "	6	"	5 200	67	98.7
Feb. 26-Mch. 8, "	11	"	3 900	42	99.0
Mch. 15-20, "	6	"	4 600	30	99.4
Mch. 26-Apr. 7, "	13	"	1 900	22	98.8
Oct. 26-Nov. 8, "	14	"	3 400	14	99.6
Feb. 7-11, 1899	5	"	6 600	140	97.9
Feb. 13-19, "	7	"	5 100	88	98.3
Feb. 24-27, "	4	"	6 800	120	98.2
Mch. 14-18, "	5	"	5 300	42	99.2
Mch. 20-Apr. 2, "	14	"	4 800	34	99.3
Apr. 6-14, "	9	"	2 400	63	97.4
Apr. 28-May 2, "	5	"	2 800	49	98.2
Nov. 15-18, "	4	"	8 400	19	99.8
Average	9	0	5 000	51	99.0
With Intermittent Operation.					
Jan. 10-14, 1898	5	3	5 200	20	99.6
Dec. 1-5, "	5	5	8 300	88	98.9
Jan. 22-Feb. 2, 1899	12	7	5 200	84	98.4
Feb. 20-23, "	4	4	7 700	82	98.9
Mch. 3-6, "	4	3	9 300	71	99.2
May 7-10, "	4	3	700	18	97.4
Nov. 5-9, "	5	3	5 400	22	99.6
Dec. 3-7, "	5	4	10 200	96	99.1
Dec. 14-18, "	5	4	15 000	42	99.7
Average	6	4	7 400	58	99.2

Comparative Prevalence of Typhoid Fever in Lawrence Before and After the Introduction of a Filtered Water Supply.—For an extended period prior to the introduction of filtered water, the death rate from typhoid fever in Lawrence was excessive. During the ten years from 1859 to 1868, the local death rate from this disease averaged 8.34 per 10 000, placing the city in this regard fourth in the list of sixteen cities of Massachusetts which introduced public water supplies in the

years from 1869 to 1877.* In 1875, as already stated, there was introduced in Lawrence a public water supply, derived to some extent from the filter gallery, but for the most part directly from the Merrimac River, and used, after storage of about two weeks, in the distributing reservoir. From 1878 to 1889 the city's death rate from typhoid fever was 8.33 per 10 000, or the second in amount obtaining in the twenty-eight places in Massachusetts which were cities in 1891, the average rate of which was only 4.62 per 10 000.* During the last four years of this term the Lawrence death rate was 10.30 per 10 000, while the average for the other cities was practically the same as for the entire period.

An inquiry made by the State Board of Health into the cause of the excessive mortality from this disease in Lawrence developed the fact that from 1889 to 1893 the prevalence of typhoid fever followed with marked uniformity outbreaks of this disease in Lowell. The sewers of that city discharge directly into the Merrimac River 9 miles above the intake of the Lawrence Water-Works.

In order to show the benefit derived from the introduction of the filtered water supply in Lawrence, so far as it may be indicated by the decreased prevalence of typhoid fever, Table No. 21 has been prepared. This table gives the number of reported cases and deaths from this disease during the six years (1887–1892) prior to the year of construction of the filter, and during the six years (1894–1899) following its completion; together with the same data for the intermediate year, 1893, and the past year, 1900. The average death rate from this disease for the State has been added for purposes of comparison.

It will be seen that the average percentage of cases resulting in death during the first period was much greater than during the second. It is significant that a study by the State Board of Health of each case of the disease resulting in death, during the years from 1894 to 1896, and in 1898, showed that during the first year in question, 12 persons, representing 50% of the deaths from the disease in the city, were known to have used for drinking purposes the polluted canal water; in 1895, 9 persons, or 56%; in 1896, 2 persons, or 20%, and in 1898, 1 person

* Extended reference to the prevalence of typhoid fever in Lawrence as compared with other cities of Massachusetts, and the early decrease in death rate due to the introduction of filtered water, may be found in the papers by Mr. Hiram F. Mills, Chairman of the Committee on Water Supply and Sewerage of the Massachusetts State Board of Health, published in the Annual Reports of the Board, as follows: "Typhoid Fever in its Relation to Water Supplies," Annual Report, 1890, pp. 525–548; "The Filter of the Water Supply of the City of Lawrence and its Results," Annual Report, 1893, pp. 545–560.

only, of the total number dying from the disease was known to have used canal water for drinking.

TABLE No. 21.—Reported Cases of Typhoid Fever and the Deaths Resulting Therefrom in Lawrence, Before and After the Introduction of a Filtered Water Supply.

Year.	Number of Cases.		Number of Deaths.		Percentage of cases resulting in death.	Typhoid fever death rate for State of Massachusetts.
	Total reported.*	Per 10 000 population.†	Total.*	Per 10 000 population.†		
Six Years Prior to Construction.						
1887	134	33	47	11.7	35	4.5§
1888	208	50	48	11.6	23	4.5§
1889	217	51	55	12.8	25	4.1§
1890	193	43	60	13.4	31	3.7§
1891	207	45	55	11.9	27	3.6§
1892	172	36	50	10.6	29	3.5§
Average	188	43	53	12.0	28	4.0
Six Years Following Construction.						
1894	92	18	24	4.8	26	3.1§
1895	79	15	16	3.1	20	2.7§
1896	42	8	10	1.9	24	2.8§
1897	72	13	9	1.6	12	2.3§
1898	85	15	8	1.4	9	2.5‖
1899	135	22	20	3.3	15	2.3‖
Average	84	15	14	2.6	18	2.6
Percentage reduction	55	65	72	78	....	35.0
1893—year of construction	141	29	39‡	8.0	28	3.1§
1900	76	12	11	1.7	16	....

*Compiled from Lawrence and Massachusetts Board of Health Reports and other sources.

†Population for years intermediate between census enumerations have been interpolated.

‡Thirty three of this number occurred during the first eight months before the introduction of filtered water in September.

§Compiled from Massachusetts State Board of Health Report for 1898, p. 807.

‖Compiled from Massachusetts State Board of Health Report for 1899, p. 726.

In order to prevent the further use of the canal water, the drinking of which had long been customary among the operatives of the various factories, notices of warning were posted, after the introduction of the filtered water supply, at all places where this polluted water was available. It is believed, however, that a considerable length of time was required to bring about among the laboring people a clear understanding of the actual danger involved in drinking the unpurified water.

The average death rate from typhoid fever in the State of Massachusetts during the groups of years considered in the table, shows that the mortality from this cause has been decreased 35 per cent. The

apparent decrease for Lawrence alone was 78%; but deducting the possible effects, as indicated by the figure above, of improved sanitary conditions, including the general introduction of public water supplies, and of better methods of medical treatment, the actual benefit derived from the use of filtered water may perhaps be fairly represented by the prevention of the deaths of 40 persons out of every 100 dying from typhoid fever in Lawrence during the six years prior to the construction of the filter.

Acknowledgment.—In conclusion, the writers wish to extend thanks to their many friends who have aided in various ways to make this description and the data contained therein exact and truthful. They desire to express their obligations to Mr. A. D. Marble, City Engineer of Lawrence, for the preparation of a plan showing the location and layout of the filter and the territory in its immediate neighborhood. To Mr. R. A. Hale, M. Am. Soc. C. E., of the Essex Company, and to the officials of the various Municipal Departments, including the Board of Health, and in particular to the Water Board and its Superintendent, Mr. A. H. Salisbury,* the writers are indebted for much valuable information. Especial gratitude is due Mr. M. F. Collins, care-taker of the filter, who, through his great interest in this study, has willingly furnished all the data and statistics at his command.

Further than this the writers desire to express their acknowledgment to the Massachusetts State Board of Health, the results of whose careful and comprehensive studies have been published in their annual reports in such detail as to make the work of compilation satisfactory and not difficult. To the designer of the filter, Mr. Hiram F. Mills, Chairman of the Committee on Water Supply and Sewerage, of the Board, the writers wish to express their obligations for friendly advice and counsel.

† Died July 5th, 1901.

DISCUSSION.

ALLEN HAZEN, M. Am. Soc. C. E.—Perhaps no point in this paper is of more general interest than the statement in regard to the rapid increase in pollution of the river water. The authors refer to the amount of sewage entering the river during the years from 1887 to 1890 as about 1 gall. in 600 of the water passing Lawrence, which had increased, as they estimate, to 1 in 200 for the year 1900, or a threefold increase in eleven or twelve years. Mr. Hazen.

At first, this estimate of the increase would seem to be excessive, but a study of various matters connected therewith indicates that it is approximately right. Table No. 22 shows the total population upon the water-shed, computed from each national census from 1850 to 1900, and also that part of the whole living in cities and towns.

TABLE No. 22.—POPULATION ON THE WATER-SHED OF THE MERRIMAC RIVER ABOVE LAWRENCE.

Year.	Total population upon water-shed.	Urban: Cities and towns over 4 000.	Rural: Towns under 4 000.	Cities and towns having sewers.	Cities and towns having purification works.	Computed relative amount of pollution.
1850	252 000	75 000	177 000	.....	.....	16 000
1860	282 000	101 000	181 000	.....	.....	19 000
1870	306 000	124 000	182 000	.....	.....	21 000
1880	351 000	176 000	175 000	138 000	.....	63 000
1890	416 000	255 000	161 000	207 000	9 000	140 000
1900	482 000	318 000	164 000	312 000	59 000	250 000

Table No. 22 also shows the population in towns having sewers at the various dates, as nearly as the speaker can obtain the figures, and the population of towns having sewage purification works. These results are shown graphically in Fig. 4.

For the purpose of estimating the relative pollution of the river at the various periods, the speaker has attempted to reduce the sewage pollution from the various classes of population to an equivalent pollution from population connected with sewers discharging directly to the river. In doing this it is necessary to make a number of estimates which, from the nature of the case, can only be roughly approximate. The first estimate is of the proportion of the population connected with the sewers in towns having sewers. Most of the larger cities upon this water-shed introduced water and constructed sewers in the decade ending with 1880. The proportion of the population connected with sewers in 1880 was comparatively small. Generally, the sewers at first reached only the more densely populated areas. Cesspools were in common use, and as some trouble and expense were

Mr. Hazen. involved in making sewer connections, the use of cesspools was only slowly discontinued. The proportion of the population connected with sewers in sewered towns has steadily increased until at the present time most of it is so connected.

For the purpose of this estimate it is assumed that 30% of the population in cities and towns having sewers was connected with the sewers in 1880; 60% in 1890, and 90% in 1900. The remainder of the population is assumed to have the same polluting effect as if living in unsewered towns. It is further assumed that the sewage effluents in towns having sewage purification works, including the chance of discharge of raw sewage, have 20% of the polluting power of the raw sewage, and that the pollution from the population in unsewered towns of 4 000 inhabitants and over is equivalent to the sewage of one-tenth, and that from rural population to one-twentieth, of that population, discharged directly. The relative pollution of the river at different times, computed in this way, is shown by the last column in Table No. 22.

These results indicate that, in a general way, during the last three decades the pollution of the river has doubled every ten years, or, in other words, the annual increase in pollution has been about 7%, a figure only a little less than that indicated by the authors' statement. This result is confirmed by analyses of the river water made by the State Board of Health. The annual average free ammonia of the river water above Lawrence is probably, on the whole, the best index, from a chemical standpoint, of the amount of sewage pollution. The figures have been as shown in Table No. 23.

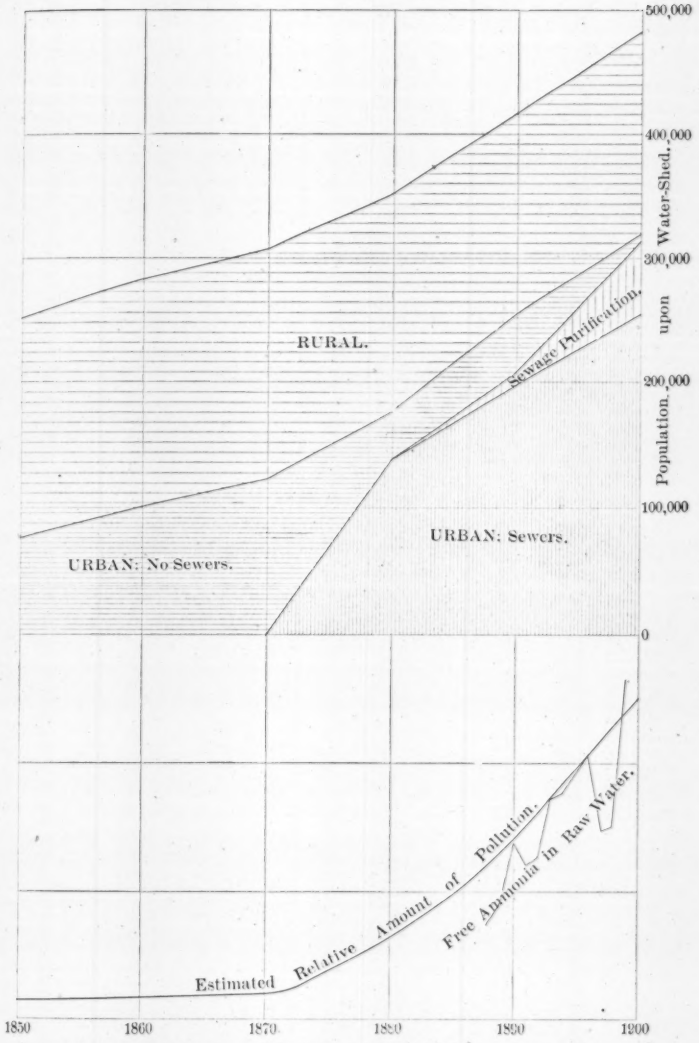
TABLE No. 23.

Year.	Annual average free ammonia in raw water at Lawrence, in parts per 100 000.	Year.	Annual average free ammonia in raw water at Lawrence, in parts per 100 000.	Year.	Annual average free ammonia in raw water at Lawrence, in parts per 100 000.
1888....	0.0026	1892....	0.0042	1896....	0.0068
1889....	0.0030	1893....	0.0057	1897....	0.0049
1890....	0.0046	1894....	0.0062	1898....	0.0050
1891....	0.0040	1895....	0.0064	1899....	0.0088

There is some fluctuation from year to year, but, on the whole, the increase corresponds substantially to the rate just indicated. This is shown by Fig. 4, in which the free ammonia is plotted in comparison with the relative pollution as just computed.

It should be said, in justice to those who were responsible for the introduction of the raw water a quarter of a century ago, that the river water was, at that time, from the present standpoint, comparatively unpolluted; and it is not improbable, that, looking at the

Mr. Hazen.



POPULATION UPON THE WATER-SHED OF THE MERRIMAC RIVER, ABOVE LAWRENCE; AND INCREASE IN AMOUNT OF POLLUTION.

FIG. 4.

Mr. Hazen. matter broadly, the introduction of a public water supply in its time was as great a step in advance as the construction of the filter at a later date.

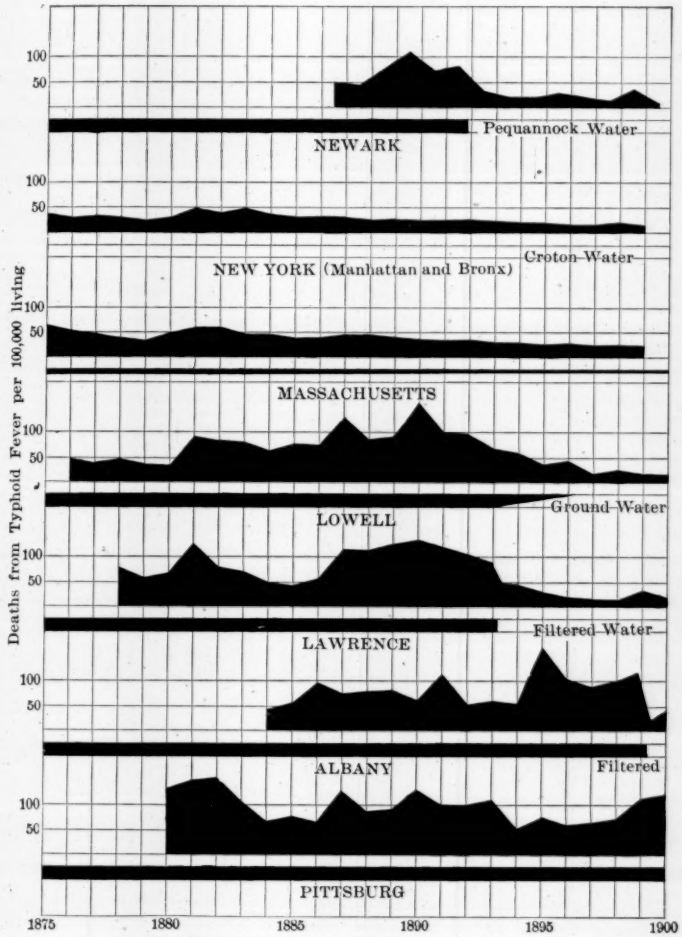
The comparison between different water-sheds, made by the authors, is interesting, but their figures need some revision. Their estimates of the growth of rural population are too great. Rural population in the Northeastern States has rarely increased in the last thirty years, and has often decreased. On the water-shed of the Hudson, above Albany, the total population, by the census of 1900, is only 80 per square mile instead of 93, as estimated by the authors.

TABLE No. 24.—DRAINAGE AREAS, AND URBAN POPULATIONS UPON THEM, OF CERTAIN RIVERS USED FOR WATER SUPPLIES.

Place.	River.	Drainage area, square miles.	Urban population upon drainage area above intake, 1900.	Population per square mile.
Chester, Pa.	Delaware.	10 450	1 810 029	173
Philadelphia, Pa.	Schuylkill.	1 915	186 682	98
Lawrence, Mass.	Merrimac.	4 634	317 869	69
Albany, N. Y.	Hudson.	8 240	376 001	46
Poughkeepsie, N. Y.	Hudson.	11 600	531 687	46
Schenectady, N. Y.	Mohawk.	3 400	142 208	42
Wilkesbarre, Pa.	Susquehanna.	9 850	315 371	32
Philadelphia, Pa.	Delaware.	8 186	242 788	30
Louisville, Ky.	Ohio.	91 000	2 308 305	25
Evansville, Ky.	Ohio.	107 000	2 560 853	24
Harrisburg, Pa.	Susquehanna.	24 030	584 672	24
Paterson, N. J.	Passaic.	773	17 205	22
Cincinnati, Ohio.	Ohio.	72 400	1 572 527	22
Pittsburg, Pa.	Monongahela.	7 600	156 412	21
Trenton, N. J.	Delaware.	6 916	145 707	21
Troy, N. Y.	Hudson.	4 570	85 864	19
Toledo, Ohio.	Maumee.	6 723	112 470	17
Pittsburg, Pa.	Allegheny.	11 400	182 332	16
Elmira, N. Y.	Chemung.	2 107	27 973	13
Grand Rapids, Mich.	Grand.	4 800	46 874	10
Quincy, Ill.	Mississippi.	135 000	1 146 173	8
Davenport, Ia.	Mississippi.	87 842	670 274	8
Washington, D. C.	Potomac.	11 476	79 563	7
Dubuque, Ia.	Mississippi.	81 322	583 884	7
New Orleans, La.	Mississippi.	1 261 084	9 162 139	7
St. Louis, Mo.	Mississippi.	700 663	4 393 016	6
Binghamton, N. Y.	N. Br. Susq.	2 279	14 780	6
Chattanooga, Tenn.	Tennessee.	21 418	61 826	3
Knoxville, Tenn.	Tennessee.	9 530	29 189	3
Richmond, Va.	James.	6 800	18 891	3
Kansas City, Mo.	Missouri.	426 893	653 667	2
Little Rock, Ark.	Arkansas.	157 466	285 507	2
St. Joseph, Mo.	Missouri.	426 000	501 902	1
Minneapolis, Minn.	Mississippi.	19 585	21 961	1
Omaha, Neb.	Missouri.	322 500	87 554	0.3

Table No. 24 shows the drainage area of rivers at points where they are used for public water supplies by some of the larger cities in the United States, and the urban populations upon these water-sheds, including all cities and towns of more than 4 000 inhabitants, by the census of 1900.

Mr. Hazen.



TYPHOID FEVER DEATH RATES AND WATER SUPPLIES.

FIG. 5.

Mr. Hazen. The urban population upon a water-shed is one of the most useful indications of the relative pollution of the water; but it must be remembered that there are many other considerations, such as velocity of flow, and amount of precipitation and evaporation, which must be taken into account; and if this could be accurately done, the order of the cities in this table might be very greatly changed. Rural population on a water-shed is a much less important source of pollution than urban population.

The cost of operation of the Lawrence filter seems excessive, when compared with figures for plants built in sections and more conveniently arranged for economical operation. It should not be forgotten, however, that this was accompanied by very low cost of construction; and when reasonable interest and sinking fund charges are taken into account, the total cost is quite moderate. The absence of preliminary pumping is an important element in this result. The results at Lawrence and at Albany may be compared, for instance, with an allowance of 5% for interest and sinking fund charges, and, using round figures, are shown in Table No. 25.

TABLE No. 25.

	Lawrence.	Albany.
Cost of plant	$70 000	$500 000
Quantity of water per annum, million gallons	1 210	4 400
Cost of operation per million gallons	$7.70	$4.75
Cost of operation per annum	$9 350	$20 900
Interest and sinking fund charges, 5%	3 500	25 000
Total cost per annum	12 850	45 900
Cost per million gallons	10.64	10.43

These figures indicate that the total cost of filtered water delivered in the pump-well, including all preliminary expenses, is substantially the same at Lawrence and at Albany.

The most striking result of the Lawrence filter has been the reduction in the city's typhoid fever death rate. In Fig. 5 this death rate is plotted from 1878 to 1900. For comparison, corresponding death rates for Newark, New York City (Manhattan and Bronx), State of Massachusetts, Lowell, Albany and Pittsburg are shown for as many years as data are available.

Under each death-rate plotting there is a band indicating the character of the water supply. Unpurified river water is indicated by a black band. Unpolluted or purified water by a light band. Under Massachusetts, the proportion of black in the band shows approximately the proportion of the total population in towns not having public water supplies, as Dr. S. W. Abbott, Secretary of the State Board of Health, has shown that in the whole State the amount of

typhoid fever has decreased in substantially the same ratio as this part of the population. Mr. Hazen.

The conditions in Lowell have been, in many respects, similar to those in Lawrence. Lowell is eight miles above Lawrence on the Merrimac River. Both cities used the Merrimac River water, while the other cities on the river did not. When the river water was first introduced the typhoid fever death rates decreased, indicating probably that the river water was, at that time, on an average, less poisonous than the wells which it superseded. Afterward, with the rapidly increasing pollution, the typhoid fever rates increased in both Lowell and Lawrence, until about 1890, when they became so high as to attract serious attention; with the result that the conclusion was reached that the river water was responsible for a large number of the deaths from typhoid fever; and measures were taken for improving or changing the supplies.

In Lawrence the city filter was constructed. In Lowell an underground source of supply was secured. At Lawrence the change was made in September, 1893, while at Lowell it extended over three years, from 1893 to 1896, as shown by Fig. 5. The reductions in the death rates in both cities were very marked. In Lawrence the reduction was more abrupt than in Lowell, corresponding to the way in which the supply was changed. Both cities now have substantially the same typhoid fever death rates, which rates are slightly lower than the average for the State.

In both cases the reduction in the typhoid rate seems to have commenced a little before the change in the water supply. The same thing has also been observed in other cases, and has been taken by many as an indication that the change in the supply was only responsible for a part of the reduction, and that much of the improvement would have been realized even though no change had been made in the water supply.

In the cases of Lowell and Lawrence, at least, the speaker thinks there is nothing to bear out such an assumption. Prior to 1890 the typhoid fever rates in both cities had increased rapidly, notwithstanding the improvements in the methods of treating the disease, and in the general sanitary conditions, to which the authors refer. The pollution of the river water was increasing rapidly, and, had no change been made in the water supplies, there is every reason to believe that, notwithstanding the continued improvements in the directions mentioned, the typhoid fever death rates would have increased, and would have been greater at the present time than ten years ago.

How, then, shall we account for the reduction just before the introduction of the new water? There may be various reasons, but two circumstances at least deserve consideration. In the first place, the typhoid fever death rate fluctuates from year to year. The conditions

Mr. Hazen. which control the dissemination of the disease by water are complicated and change with conditions of weather, stream flow, etc. Generally, the highest rates occur at intervals of several years, between which lower rates are observed. An unusual increase in typhoid fever attracts attention, excites public interest, and results in the determination to get a new source of water supply. The new supply is generally introduced a few years later, when, in the natural course of events, there has been a decline in the typhoid fever death rate from the summit, and before the inevitable increase to another summit has begun. A second reason for the decline prior to the introduction of the new water is that the official recognition of the injurious character of the water, which is involved in the determination to secure another source of supply, makes the inhabitants more careful in the use of the existing supply, and in this way tends to reduce its effect upon the death rate.

The reductions at Lawrence and Lowell for the years 1890 to 1892 were hardly more striking than corresponding reductions at Albany and Pittsburg, shown in Fig 5; but there is this difference, that at Lawrence and Lowell, with a new water supply, the reduction continued at an accelerated rate, while at Albany and Pittsburg the reduction did not continue, but on the contrary, rates have since occurred as high or higher than those previously experienced. In speaking of Albany, reference is made only to the period prior to the introduction of filtered water. Since that time, the death rate has been much lower.

An interesting comparison is furnished by Newark, which introduced in 1892 a gravity supply from unpolluted sources in place of the Passaic River water previously used. The reduction in the typhoid death rate at Newark was almost as striking as at Lawrence and Lowell.

In view of the foregoing, the speaker believes that the authors' estimate of the amount of reduction due to the construction of the filter is far below the truth; and that in case the water supplies of Lowell and Lawrence had remained as they were in 1890, the death rates would have increased instead of decreasing as they have done. The introduction of a new water supply in each case resulted in a radical reduction in the typhoid fever death rate, and the reduction at Lawrence, where filtered water from a very highly polluted source was used, was quite as great, and the subsequent death rate has been as low, as at Lowell, where the new supply was a ground-water, which can hardly be supposed to carry any germs of disease, and as at Newark, where the new supply is from unpolluted sources.

While the most striking reduction in the death rate following the use of filtered water was in typhoid fever, the general death rate showed a very considerable reduction. Table No. 26 shows the death

rate per thousand living for the seven years that the filter has been in operation, together with the corresponding rates for three preceding periods of seven years each. Mr. Hazen.

TABLE No. 26.—DEATH RATES IN LAWRENCE BY SEVEN-YEAR PERIODS, 1873 TO 1900.

Year.	Total deaths per 1000.	Year.	Total deaths per 1000.	Year.	Total deaths per 1000.	Year.	Total deaths per 1000.
1873.....	20.6	1880.....	21.7	1887.....	22.5	1894.....	19.0
1874.....	21.7	1881.....	20.4	1888.....	19.1	1895.....	20.3
1875.....	26.0	1882.....	22.3	1889.....	25.7	1896.....	20.3
1876.....	22.3	1883.....	21.8	1890.....	26.6	1897.....	20.6
1877.....	22.7	1884.....	22.9	1891.....	24.5	1898.....	19.7
1878.....	22.2	1885.....	19.9	1892.....	26.2	1899.....	20.5
1879.....	20.8	1886.....	20.0	1893.....	24.2	1900.....	20.0
Average..	22.3	.....	21.3	.....	24.1	.....	20.1

The reduction in the general death rate in the last seven years, as compared with the seven preceding years, is several times as large as the reduction in the typhoid fever death rate, and perhaps indicates the effect of the quality of the water on other diseases. The question as to how far this is due to the water supply, and how far to improved general sanitary conditions, may be properly raised; but the very decided reduction at the time of the introduction of the filter, and the steadiness with which the rate has continued on a lower level since, seem to indicate that the water supply had something to do with the reduction, and that the coincidence was not accidental.

There is a particularly interesting history connected with this filter, which, unfortunately, is not included in the paper. This history relates to repairs made to the underdrainage system in 1898. These repairs were made necessary because there was no water-tight bottom to the filter, and a certain amount of ground-water entered from below, and this ground-water carried iron, and where the iron-containing ground-water came in contact with air, or aerated water, conditions were formed which allowed the growth of a plant called *Crenothrix*. The growth of *Crenothrix* took place about and in the underdrains, and gradually closed them. It became evident that unless something was done, a sufficient supply of water could not be secured. To correct this it became necessary to dig up and reconstruct the drainage system. As the filter is all in one unit, the work had to be done during the short intervals when the filter was drained; and it is perhaps not surprising that, owing to the disturbance, the bacterial efficiency fell considerably, namely to 97.7% for November, and to 94.3% for December, 1898, after which the efficiency rapidly regained its ordinary standard. This decreased efficiency was followed early in 1899 by a very con-

Mr. Hazen. siderable increase in the number of cases of typhoid fever in the city, and the deaths from this cause for the year were about double, or ten more than the average for the years before and after.

On the face of it, and in the absence of other explanation, it seems probable that this increase in the typhoid fever death rate was the price which the City of Lawrence paid for repairing the filter while maintaining the supply from it; and the speaker thinks that this experience may teach one of the most useful lessons to be found in the history of the Lawrence filter. It shows, as a number of corresponding experiences in Europe showed many years ago, that a very slight falling off in the efficiency of filtration may affect to a considerable extent the health of the people using the water; and that the death rate of the city is decidedly a function of the number of bacteria remaining in the filtered water, even though that number is comparatively small.

The question may be raised as to whether even the best possible filtration removes all the typhoid germs from the water, and whether some of the remaining typhoid fever in the city is not caused by the filtered water, even under the most perfect conditions. Present evidence is certainly not conclusive on this point. The most that can be said is that such a reduction in the number of bacteria as is caused by the best filtration reduces very largely the death rate from typhoid fever among the users of the water; and that water so filtered, even though taken from much-polluted sources, compares favorably from a hygienic standpoint with the best water supplies otherwise obtained. No certain evidence can be produced that typhoid fever is caused by waters filtered by what may be considered the best methods, nor, from the nature of the case, can evidence be produced that no typhoid fever is caused by them; but it is known that if any is caused, the amount must be extremely small.

This experience at Lawrence should warn us that it is not safe to regard as unimportant the numbers of bacteria passing a filter, and to regard one percentage of removal as good as another as long as all the percentages are above some arbitrary limit; and in the present state of the art the speaker thinks that we should not be content with anything less than the highest bacterial efficiency which can be obtained by any practicable means.

The authors' statement that the filter was constructed according to plans and directions given by the State Board of Health is, the speaker believes, not strictly correct. The plans were not furnished by the Board, but by Mr. Hiram F. Mills, a member of the Board, who acted in this connection entirely as a public-spirited citizen, and not officially. Mr. Mills gave his services to the city and received no compensation for them.

Mr. Mills' personal interest in the filter should not go without

notice. He was Chief Engineer of the Essex Company. The Essex Company had created Lawrence, and was, perhaps, more directly interested in its welfare than any other corporation or individual. It is hardly possible to speak too highly of Mr. Mills' interest in the city, and in securing for it that which was best. The speaker does not overstate the case in saying that the construction of the Lawrence filter was solely due to the untiring and well-directed efforts of Mr. Mills, who met and overcame all the objections which were raised to it, and carried the project through successfully. Mr. Hazen.

The Lawrence filter has resulted in saving many hundred lives and preventing thousands of cases of sickness. Its original purpose has been splendidly achieved, and all the expenditures involved would have been fully justified even though they had been many times greater than they have been. The lessons that can be learned from its history are many; but, perhaps, none is more important than the courageous application of principles deduced from careful experiment and well-founded theory, but not yet substantiated by practice.

The Lawrence filter was in a large sense the beginning of water purification in America. This is particularly true when the matter is considered from a hygienic standpoint. A few filter plants had been built in America before the Lawrence filter, but these earlier plants had been directed largely or principally toward improving the physical character of the water. At Lawrence, for the first time, a filter was constructed with the deliberate purpose of reducing the death rate. The success that was achieved in accomplishing this result attracted the attention of the whole country, and was of the greatest service in directing attention and energy to a most important subject, which up to that time had not received the attention which it deserved.

ROBERT SPURR WESTON,* Esq. (by letter).—The authors have certainly done much to advance the knowledge of the subject of water purification by this complete compilation of data connected with the Lawrence filter, the classic of its type, in America. Mr. Weston.

The educational value of this paper must be very high, as the authors have faithfully recorded the various steps which have been taken since the inception of the work and in the direction of better and more economical operation.

The paper has a value in helping us to realize how much sanitary science owes to Mr. Hiram F. Mills and his associates who gave freely of their time and knowledge, in spite of local distrust, and, on a small appropriation, built a filter which has stood so well the test of over seven years of daily operation.

The authors have treated their subject so carefully and so judicially that there is little to be said in criticism.

* Resident Expert, Water Purification Station, New Orleans, La.

Mr. Weston. They have brought out, among others, the following important points:

a. The efficiency of the filter, as made evident by the improved health of the community.

b. The permanency of the results obtained.

c. The trouble due to entrainment of air, caused by filling the filter rapidly from above.

d. The efficiency of continuous, as compared with intermittent, filtration.

e. The lack of economy shown in not building covered filters and in not equipping the plant better generally.

f. The economies effected in the cost of scraping, sand washing, etc.

They have also succeeded in bringing to light many useful and hitherto inaccessible data as to cost.

The writer would like to ask whether or not there were any difficulties experienced on account of the character of the bottom of the filter, which, he has always understood, was not made absolutely water-tight?

The average total hardness of the Potomac water, as determined by the writer, is 93 parts per million; this figure might fill the blank in Table No. 3.

Mr. Bailey. GEORGE I. BAILEY, M. Am. Soc. C. E.—Students of water purification have long been in debt to the Lawrence, Mass., water filter, for the work it has done, and to the Massachusetts State Board of Health for making known the results obtained. This debt is now further increased by the very comprehensive and complete descriptions and details of operation and cost given by the authors of this paper.

This plant does not now represent the best practice, either in design or in operation, and the lessons of what to avoid are indicated by the details of cost given. A few of these points seem to be:

1.—The desirability of subdividing the filter area into two or more units, of such size that one unit may be taken out of use for cleaning or refilling with sand, without embarrassing the general operation or the water supply.

2.—The desirability of covering filters, where the winter temperature approaches that at Lawrence.

3.—The substitution of other than manual labor, where it is possible and feasible, to carry out the maintenance operations.

To make comparisons along these lines, in addition to the data given in this paper, there are available the records of the Poughkeepsie filter,* the operation expense for the Mount Vernon filter,† and also the operating cost of the Albany filters.

* Fifth Annual Report of the Board of Public Works, Poughkeepsie, N. Y., for the year 1900.

† *Engineering News*, May 30th, 1901.

Mr. Bailey.

In Table No. 27 these records are compared with those of the Lawrence filter for the year 1900, except that the Albany records are for the sixteen months' operation from September, 1899, to Decen.ber 29th, 1900.

Table No. 27 is arranged as near as may be by corresponding items.

TABLE No. 27.—Cost per Million Gallons Filtered.

Lawrence.		Poughkeepsie.		Mount Vernon.		Albany.	
Scraping	$1.75	Cleaning beds	$1.31	Cleaning	$1.31	Scraping and Wheeling out	$0.69
Sanding	1.02	Washing and replacing sand	0.82	Overhaul	1.43	Refilling	0.32
Washing	1.25	Labor, care of grounds, tools, etc	1.59	Washing	0.75	Washing	0.42
General	0.60			Miscellaneous	0.10	Incidentals and lost time	0.23
Removing ice	1.92		0.60		0.23		
Conveying	1.16			New sand	0.23		
Totals	$7.70		$4.32		$4.05		$1.66

The filters at the first three places are uncovered, at Albany they are covered.

The number of units in the plants are, at Lawrence 1, at Poughkeepsie 3, at Mount Vernon 3, at Albany 8.

Direct comparisons of these costs cannot be made, as each city has some item not included in the others, and the cost is upon a different basis, as follows:

At Lawrence, laborers are paid $2.00 for nine hours' work. At Albany and Poughkeepsie, the pay is $1.50 for eight hours' work. Lawrence, therefore, pays 18.5% more for labor. It is not exactly correct to reduce Lawrence's gross cost by this amount, but, as nearly all the operating cost is for labor, in so reducing it a comparative basis is more nearly obtained. The item "Conveying" is for work not necessary to the same extent at the other plants, and is probably due to the location of this individual plant and not to filtration generally.

At Poughkeepsie 1 000 cu. yds. of sand were scraped from the beds. Only 910 cu. yds. were washed and replaced. To round out the cost, an additional amount of $0.07 should be added. The quantity of water used in washing sand by their method is estimated to be 18 to 1. The cost of this water is not included, and, on the basis of their pumping expenses, $0.10 should be added for this.

In Mount Vernon no cost is added for wash water. Their washer is approximately the same as at Lawrence and Albany. This allowance should therefore be about $0.05. In their cost no charge is made for maintenance of grounds, that being charged to "Pumping Station." Without any definite information, it is assumed that this cost is approximately $0.20 per million gallons. In their exhibit there is an

Mr. Bailey. item of $0.23 for new sand, which should be omitted, as it should be charged to construction, or, if it is to make up for sand lost in washing, it should be distributed through all the years of operation.

TABLE No. 28.—ADJUSTED COMPARISONS OF COST PER MILLION GALLONS FILTERED.

	LAWRENCE.	POUGHKEEPSIE.	MOUNT VERNON.	ALBANY.
Scraping	$1.43	$1.31	$1.31	$0.69
Sanding	0.84	0.98	1.41	0.32
Washing	1.02		0.37	0.42
Removing ice	1.57	0.60	0.23	0.61
General	0.49	1.59	0.30	0.23
Conveying	0.95	.....	.....	.....
Total	$6.30	$4.48	$3.62	$2.27

At Albany, the filters are covered. If this were done only to prevent ice formation and ice removal, to compare with other plants, the cost of the covering, say, $15 000 per acre, at 3½% for sixteen months, and this were divided by the quantity filtered, the charge for ice removal would be $0.61.

By recalculating a few other items, to reduce them to a common basis, and taking into account all of the foregoing, the ordinary cost of filtering by this method would be as shown in Table No. 28.

The basis of all operation of sand filters is the work done with the filtering sand. Its handling, its scraping, its moving out, washing and replacing in the bed, and the conveniences and facilities for handling the sand, or the lack of them, make up the cost. It is interesting to note that the cost of each of these plants is almost directly proportional to the amount of sand handled, as is shown in Table No. 29.

TABLE No. 29.

	Millions of gallons filtered.	Sand handled. Total cubic yards.	Sand, in cubic yards per million gallons filtered.	Proportional percentage.	Cost.	
					Percentage of sand.	Actual.
Lawrence	1210.	3 000	2.48	39.4	$6.30	$6.38
Poughkeepsie	653.3	1 000	1.53	24.3	4.48	3.94
Mount Vernon	679.4	955*	1.41	22.4	3.65	3.63
Albany	6463.	5 673	0.88	14.	2.27	2.27
Totals	.....	.....	6.30	100.	....	....

* New sand deducted.

The water at Albany is more turbid, and contains more suspended matter than at either Lawrence or Poughkeepsie, and it is presumed

more than at Mount Vernon. Ordinarily, this would involve a greater cost of operation at Albany than at any of the other cities. Generally speaking, the turbidity and suspended matter in the water are the dominant factors which control the selection of a method of filtration, and the numerous investigations that have occurred, and are now being had, pay chiefest attention to these items. If all waters were reasonably free from these properties, there would be little discussion as to the method of filtration to be used, except possibly for the removal of color. Mr. Bailey.

At Lawrence, Poughkeepsie and Albany the work is carried on by a municipal corporation, with all the political influences which tend to add to the cost of work. At Mount Vernon it would be expected that, as the filter is controlled by a private company, the work should be more economically done, and, to some extent, the figures bear this out.

Taking into account these different factors, it is seen that the costs at Lawrence, Poughkeepsie and Mount Vernon are greater than at Albany, although it might be expected that the cost at Albany would be the greatest. Each of the items of handling sand is less at Albany than at the others, and, in the speaker's opinion, this is a clear indication that the effect of the covers is felt throughout and upon all operations. The cost is greatest at the plant of a single unit.

A decided benefit in covering filters is the maintenance of the laborer's organization. These men are paid only when they work, and they want fairly steady work or they will seek other employment. It pays the management to retain experienced men, for, as they become broken in to the work, they will do nearly double the amount done by inexperienced men. With covers on the filters, the men scrape a filter during any storm; therefore, a rainy day means no loss of wages. They remove the scraped material, for the long run of the barrow, and its filling, is done under cover, and they do not object to a slight wetting. In winter they are shielded from winds and storms; the temperature inside the filter is less than outside, and the men make full time, irrespective of weather, and without any special discomfort.

These benefits of covers are entirely without consideration of any climatic or thermal conditions regarding the efficiency of filters. Such benefits have been so often mentioned, that it is useless to mention them here.

There is one cost of filtering, at a plant having only one unit, or which is badly subdivided, which does not show in the filter records, but appears only in the pumping cost. Where the entire filter area is of a size to yield daily only the quantity of water needed, and it is all in one unit, the cost of pumping is increased at every cleaning, as the pumps are necessarily shut down and the fires banked or put out. The pay of the employees goes on, but no work has been done for the money, and the only decrease in cost is the slight amount of coal and

Mr. Bailey. oil saved by not operating the pumps. With the filter area subdivided, and only one unit out at a time, either the operating portion may be forced to yield the desired quantity of water, or else the only loss, in addition to the lessened quantity, will be a decreased duty of the pumping engines. The loss in efficiency of the filter, due to a moderately increased rate of flow, will not be material, provided proper judgment and care is, and has been, used in starting and operating the beds.

At Albany it has been found to be a mistake to not entirely drain a bed before cleaning. The water should be completely off the sand surface, and, in the slight depressions which always occur in the surface, no puddles should be left. With a dry surface, the cleaning is done more quickly and more thoroughly. With a wet bed, the men are apt to slight the surface underneath the water, and the escape of the water from the shovel as it is lifted, carries a portion of mud and slime with it, which runs down into the sand, and the result is a bed which is unclean in spots and which will cause uneven rates of filtration. A bed thus scraped always makes a short run between scrapings.

All the plants named are defective, in that they depend upon manual labor for all handling of sand between scraping the bed and refilling it. There is no good reason why a movable ejector hopper should not be utilized, and, after the sand is scraped into piles, it should be shoveled into this hopper, and thereafter not again touched with the shovel or barrow until it is taken away from the sand washer, or deposited upon the bed. It is known that experiments are being made, and such an arrangement will be incorporated in some of the filters now contemplated or being designed, and the effect will be to reduce materially the cost of operating sand filters.

Mr. Wachter. LEONARD M. WACHTER, Esq.* (by letter).—In comparison with the quantities of water filtered at Lawrence between successive scrapings, it is thought that the following facts and the diagram (Fig. 6) showing what has been done at Albany under the conditions met there will be of interest.

The water of the Hudson at the Albany plant is comparatively clear for, probably, at least 75% of the time. During the year 1900 there were 71 days when the turbidity was 0.100 and over (wire-method, reciprocal scale). Turbidity reaching this figure is caused by heavy rainfall over a large portion of the water-shed, and by thaws and spring freshets. The suspended matters consist of silt and, also, considerable clay at times.

Table No. 30 shows the length of service, quantity filtered per acre, and turbidity of the raw water for Filter No. 8, from the start, July 27th, 1899, until May 18th, 1901.

That it is a fair average, is shown by the following: The average

* Chemist in charge of Albany Filter-Plant.

Mr. Wachter.

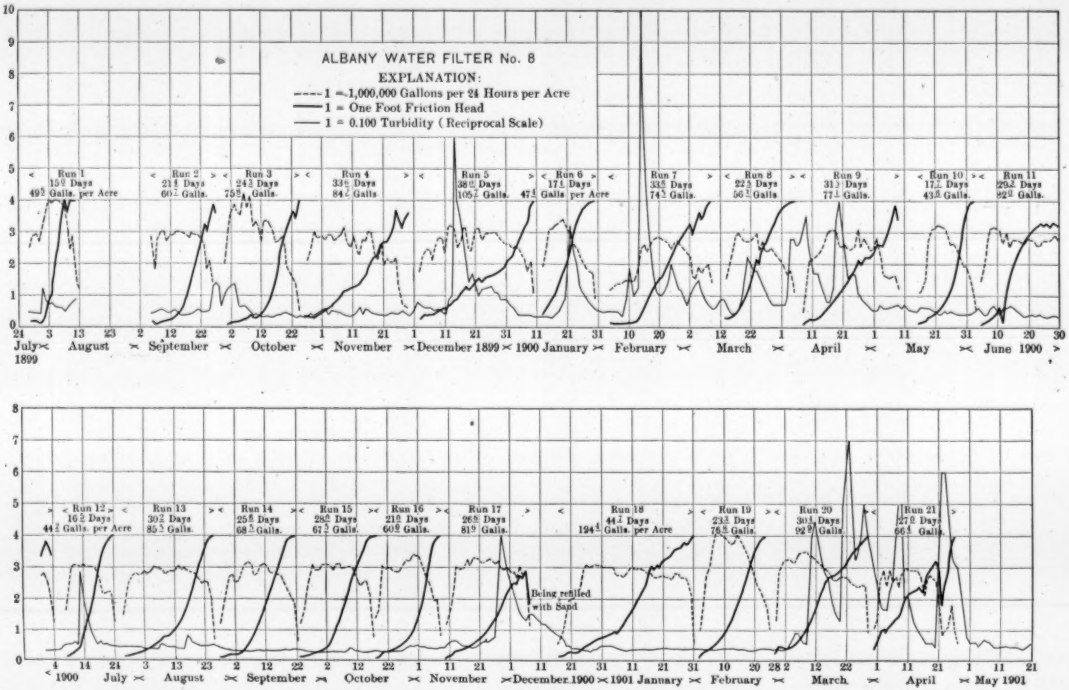


FIG. 6.

Mr. Wachter. length of runs for all filters up to December 29th, 1900, was 25.8 days. The average quantity filtered per acre between scrapings was 70 000 000 galls. For the filter tabulated the average length of run was 25.9 days. The average quantity filtered per acre was 71 200 000 galls. The average rate while in operation was 2 750 000 galls. per acre per twenty-four hours.

TABLE No. 30.

Number of run.	Date of starting.	Date of end of run.	Length of run, in days.	Millions of gallons per acre.	Maximum turbidity.	Number of days during which the turbidity was 0.100 or over.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	July 27	Aug. 12	15.0	49.5	0.120	1
2	Sept. 6	Sept. 27	21.4	60.7	0.140	2
3	Sept. 30	Oct. 22	24.3	75.8	0.130	3
4	Oct. 27	Nov. 29	33.6	84.2	none	none
5	Dec. 3	Jan. 10	38.0	105.8	0.600	16
6	Jan. 13	Jan. 30	17.5	47.4	0.320	7
7	Feb. 4	March 9	33.6	74.5	0.900	24
8	March 14	April 5	22.5	56.3	0.280	10
9	April 8	May 9	31.5	77.0	0.400	18
10	May 16	June 4	17.7	43.6	none	none
11	June 5	July 4	29.3	81.9	none	none
12	July 8	July 24	16.5	44.2	0.290	3
13	July 27	Aug. 26	30.2	85.5	none	none
14	Aug. 28	Sept. 22	25.4	68.5	none	none
15	Sept. 23	Oct. 17	23.7	67.5	none	none
16	Oct. 18	Nov. 8	20.8	59.9	none	none
17	Nov. 10	Dec. 7	26.6	81.6	0.400	11
18	Dec. 17	Jan. 31	44.7	124.4	none	none
19	Feb. 2	Feb. 25	23.3	76.8	none	none
20	Feb. 27	March 29	30.4	92.9	0.700	20
21	March 31	April 27	27.0	66.4	0.600	21
22	April 30	May 18	17.5	41.2	none	none

Runs Nos. 7, 9 and 20 (see Fig. 6) were during periods of the highest turbidity of quite long duration, and delivered above the average quantities between cleanings, while Runs Nos. 10 and 22, following periods of high turbidity, are two of the lowest quantities filtered. This would seem to show that the yield of runs succeeding periods of high turbidity is materially reduced, while the runs during periods of high turbidity are not noticeably shortened. Observations of the condition of the beds at such times show that the mud penetrates the sand in considerable quantities for a distance of about 2 ins., and, as it is not practicable to remove this amount of sand at one scraping, it evidently forms the nucleus for resistance, and the yield of the succeeding run is reduced.

Run No. 12 was probably reduced by micro-organisms in the raw water, as the time of the year was favorable.

Run No. 17 was not completed. It was necessary to scrape the

filter so as to refill sand into the bed to clear the court yard before the advent of winter. Mr. Wachter.

Run No. 18 was directly after refilling. and this fact, with the absence of high turbidity and the favorable season of the year, made this the largest yield from any filter.

M. F. COLLINS,* Esq.—Inasmuch as certain criticisms have been made at various times in regard to the excessive cost of operating the Lawrence City filter, as compared with the expense of maintenance of sand filters at other places, it has seemed fair that the conditions producing this result should be given in some detail. Mr. Collins.

The speaker does not intend to discuss general matters of interest, but will simply confine his remarks to a statement of some of the conditions of operation which have occurred at Lawrence. Some of these are entirely local, and are due to the arrangement of the filter plant, while others, such as non-covering and climate, appear to exercise a greater influence here than at most other places where filters are in operation.

CONDITIONS DUE TO OPEN SURFACE.

Disturbance of Work by Rain.—During the period that the speaker has been in charge of the filter. he has found that the occurrence of rain storms plays an important part in increasing the cost of maintenance. Occasionally, when draining the filter during the night, in order to have it ready for scraping the next day, a rain storm has come up and has effectually prevented all work, the yield of filtered water meanwhile being entirely cut off and so remaining until the weather had cleared and the filter had been scraped and again placed in service.

Moreover, it has sometimes happened that while scraping the filter with a large working force, a heavy shower, or possibly a more extended rainstorm, has suddenly blown up before the piles of scrapings had been wheeled out. This has caused an abandonment of the work, the laborers being retained under pay with the expectation of an immediate clearing in the weather, which might or might not quickly happen, causing an increased cost of scraping. Occasionally, the necessity of securing water has compelled the men to return to work, notwithstanding the rain, if this is long continued. This latter condition is of itself extravagant, inasmuch as the sand, while wet, is in all ways much more difficult to handle. It will be seen, therefore, that rainy weather during times of scraping necessitates an increased cost of this operation.

Necessity of Removing Snow and Ice.—When extreme cold weather sets in, the whole area of the water surface freezes, oftentimes to depths of from 8 to 12 ins. In order to scrape a section of the filter at such

* Superintendent of Lawrence Water Department.

Mr. Collins. a time, the ice, together with any snow which may be upon it, must first be cut and removed, usually on the day before draining. In cold weather an additional coating of ice, sometimes 1 in. or more in thickness, forms before the filter is entirely drained, causing an increased cost and delay in scraping.

Difficulty Caused by Thin Ice on Sand Surface.—After a section has been made free from ice, and draining is being carried on, a thin ice layer forms upon the water surface during cold weather, and, lowering with the water, descends to the sand surface and becomes frozen to it. Scraping is thus rendered very difficult, and the amount of material removed is largely increased, on account of the ice and the necessity of scraping deeper. Those who have had experience in cleaning filters of this sort can readily appreciate the trouble from these conditions, and the increase in expense of scraping caused thereby.

Difficulty Caused by Snow.—During the winter serious difficulty in scraping and conveying is caused by light falls of snow. However light the snowfall, if the sand surface is covered, the depth to which scraping should be carried on cannot be gauged as accurately as in clear weather, and it has been found by the speaker that there is invariably a tendency to scrape too deeply with the shovel. This circumstance is very trying when every effort is being made to reduce the cost of maintenance, but there seems to be no method of preventing it, inasmuch as laborers require long experience in this work to enable them to know to what depth to carry the shovel during the various conditions of scraping.

Difficulty Caused by Freezing of Sand Surface.—During the early and late portions of the winter, even when the weather is not cold enough to freeze the water surface to any extent, it sometimes is capable of producing frost in the filter sand after draining to a considerable depth. Even a depth of freezing not greater than 1 in. causes an increase in the amount of sand which it is necessary to remove, and consequently an increase in the cost of scraping.

Conditions Arising from the Lack of a Convenient Storage Place for Scraped Sand.

Conveying sand from the filter is always an expensive operation, but especially so in winter, on account of the lack of a convenient place in which to store the scraped material after it has been removed from the bed.

The lower roadway bordering upon the north side of the filter is utilized as a temporary dumping ground, where sand remains over night or until it can be removed during the following day or days to the permanent place assigned for dirty sand in the neighborhood of the washing machine. During cold weather this deposit of sand becomes frozen, and extra labor is required to break it up and get it

into condition for removal to the permanent dump. This double handling is, of course, expensive, but seems to be necessary under present conditions, due to the peculiar location of the filter. With suitable conveying apparatus the cost of this item of maintenance could doubtless be reduced in very large measure. Mr. Collins.

CONDITIONS DUE TO THE CONSTRUCTION OF THE FILTER IN ONE UNIT.

In the last annual report of the Lawrence Water Board, for the year 1900, the speaker recommended that two transverse division walls be placed in the filter, making three filtering sections of about equal area. If this work be done it will greatly facilitate the operation of the filter, and thereby lessen the cost of maintenance. One section may be cleaned at a time, while the other two may be operated as usual. With this construction, the comparatively small force of men now required for the routine work of maintenance would also be sufficient for the performance of scraping, which it has been shown is not now the case. Moreover, the permanent force of laborers will individually become more skilful in performing all the operations devolving upon it. The importance of this last feature can scarcely be over-estimated. In the process of scraping alone a great saving can be effected, as green men usually go from ½ to 1 in. deeper than is necessary in the process of scraping. It will be seen that the effect of the increased quantity of material thus removed is to increase, not only the cost of scraping, but also of conveying, washing and sanding.

The speaker has thus briefly outlined some of the leading obstacles which occur in the operation of open sand filters, constructed as a single unit, in a climate where the winters are variable and often severe. He trusts that these suggestions may be instrumental in bringing out a discussion of similar problems confronting the practical operations of other filtration works, and that they may serve to suggest opportunities for improvement in designing other plants.

GEORGE W. FULLER, Assoc. M. Am. Soc. C. E.—This paper is a welcome addition to the literature upon water purification, in that it brings together for ready reference the leading information from various reports and records relative to the Lawrence filter, and especially in that it places on record more comprehensive data concerning the operation of the plant than have been readily available hitherto. Mr. G. W. Fuller.

This filter holds a unique position among municipal works, and engages especial attention, not only from engineers, but from workers and students in various sanitary lines. To appreciate fully the position which the Lawrence filter occupies, it is necessary to consider with some care the conditions which were associated with its installation. Looking back nine years, and from the standpoint of our present knowledge, it might seem at first sight that there was nothing remark-

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able about the installation of a plant for the purification of a municipal water supply amounting to about 3 000 000 galls. daily.

In 1892, when the construction of this plant was begun, however, the entire subject of water purification was in a state much less clearly crystallized than at the present time. It is true that important studies had been made in the application of the germ theory of disease to problems in epidemiology, and also in the application of bacteriology to questions of water purification. At the Lawrence Experiment Station, furthermore, there had been in operation for some months a series of experimental sand filters for the purification of the Merrimac River water, from the studies of which were discovered some of the more important laws of filtration under the conditions which there existed.

In spite of all this, it was a period of much uncertainty and of many marked differences of opinion. Comparatively little progress of a substantial nature had been made for some time in the field of water purification in Europe, and in the United States the subject had laid almost dormant for twenty years, so far as the installation of municipal plants was concerned. The technical journals of Europe had much to say concerning the outbreak of typhoid fever during the winter of 1889 in the eastern portion of the City of Berlin, where the water consumers were supplied with filtered water from the old Stralau sand filters, which were operated on the continuous method. At London a Royal Commission was engaged in an investigation of the future supply for the Metropolitan District, and received testimony from prominent professional men to the effect that the filtered Thames water should be abandoned and replaced by new supplies from mountain sources. In a number of the larger cities of Europe vigorous discussions were occurring with regard to the urgent need of providing municipal supplies only from unpolluted mountain streams or from ground-water. In this country a few scattering mechanical filter plants had been installed; but the available evidence as to their hygienic efficiency was not favorable. Sanitarians were disposed to give to filtration in general less credit than it really deserved, and engineers were sceptical as to its practicability in many cases on the ground of its cost.

In the Merrimac Valley several of the larger cities, including Lawrence itself, had for several years suffered severely from epidemics of typhoid fever. It will also be recalled that late in the summer of 1892 Asiatic cholera appeared in New York Harbor. Under these experiences and conditions there was full recognition on the part of the Lawrence city government that cholera might become established temporarily in the cities located above them in the Merrimac Valley, and that, if so, their water supply must either be purified or cut off. They acted wisely and promptly in placing at the disposal of the Water Board such sums of money from their limited available means as

would enable to be carried out the recommendations of the State Board of Health, whose advice they had sought. Mr. G. W. Fuller.

It is said that it is frequently darkest just before the dawn, and subsequent developments show that such, in a measure, was the case in the instance not only of the Lawrence water supply, but of the entire subject of pure water and of water purification throughout this country. It is difficult to estimate readily how great has been the benefit which this filter has produced outside of the City of Lawrence. It has proved a stimulus and an example for many later developments in this field, and it has served as a basis of reference for engineers and sanitarians, not only in this country, but also in Europe. No matter how many filters may be built in years to come, which are larger in capacity, or more efficient or more economical in operation, it is very doubtful if any will succeed in displacing the Lawrence filter from the classical position which it now securely holds as marking the beginning of a new epoch, and in fact the first epoch of important practical accomplishments, in this country, in water purification.

The authors of this paper commence their description of the filter with the statement that it was designed by Mr. Hiram F. Mills, Chairman of the Committee on Water Supply and Sewerage of the State Board of Health of Massachusetts. To the thought of the speaker, this hardly tells the whole story. It is true that the State Board of Health advised the city government with regard to the adoption of a filter, and that its construction was commenced under the general direction of the Board. Mr. Mills, however, in addition to his official connection with this project through the State Board of Health, also served in his individual capacity as Consulting Engineer to the Lawrence Water Board. Without compensation he designed this filter, based in a large measure upon the studies made at the Lawrence Experiment Station, and took the greatest interest in the carrying out of the project, even to the extent of loaning to the city some of his own assistants and foremen to aid in the work. In discussing the Lawrence filter the speaker cannot refrain from calling attention to the great debt of gratitude which is owed to this most distinguished engineer and public-spirited citizen, by the water consumers, not only of the City of Lawrence, but also of the entire country. It is to him, more than to any one else, that credit is due for installing and carrying on judiciously planned and carefully studied investigations, which have paved the way for many recent improvements in this branch of public service.

One of the most characteristic features in the design of the Lawrence filter consists in the fact that the entire filtering surface is unbroken by any division walls, such as are provided in practically every other municipal filter, in order that a portion of the plant may be thrown out of operation for cleaning or repairs, while the remain

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ing portion continues in service. This arrangement at Lawrence was made possible by the size of the plant, and by the fact that the filter was to be operated intermittently, a method which at that time Mr. Mills thought to be superior to continuous filtration.

In order to secure reasonably uniform rates of filtration through recently scraped portions of the surface, as compared with the remainder of the filter, the sand and gravel layers were designed in a most ingenious manner, as is shown in Fig. 2. This arrangement of the gravel layers, together with the use of a sand layer of different sizes and of varying thickness, according to the distance from the collecting pipe, obviously restricts the flow of water which would naturally pass through the area of clean sand surface of any one of the 25 sections that form the filter. It was provided in the design that the loss of head in this filter should not normally exceed about 2 ft., and that a portion of the filtering surface should be scraped each day, the area of that portion being such that the entire surface would be scraped about once a month. Under these conditions the restrictions in the exit passages for the filtered water were designed to be such that it would be impossible to secure a rate of filtration of more than about 5 000 000 or 6 000 000 galls. per acre per 24 hours through a recently-scraped portion of the filter surface. Further, as only about one-twenty-fifth of the entire surface would be scraped at a time, under ordinary circumstances, as originally provided for, the comparatively slight deterioration in the water filtered at this increased rate would be very largely offset by dilution with the filtered water coming under fairly normal conditions from the remaining portion of the filter. Thus, the filter was designed to serve as its own controller, within certain limits, and to offset by itself the results of differences in the amount of sediment deposited upon the surface. This feature is mentioned because it seems to be a point which is not very widely appreciated.

As is generally known, the Lawrence filter, for various reasons, has not been regularly operated in accordance with the original instructions, and in fact during the winter season it is almost impossible to carry out exactly the normal plan of procedure in the absence of a cover. Notwithstanding this, the results as to the quality of the filtered water are such as to show that as a general proposition Mr. Mills' design is a successful one for the purpose for which it was intended. Taking up the matter in specific terms, practically no data have been published to show how uniform the rate of filtration is over the different portions of the filter which have been cleaned at various times. About the only point which is known, in addition to the general success of the arrangement, refers to the somewhat higher rates of filtration which follow scraping in that portion of the filter which is nearest to the pumping station.

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How much of a factor the presence of ground-water or of water filtered through the bottom from the river has been with reference to the clogging of the underdrains, due more or less to dissolved iron, is unknown to the speaker. Taken in connection with the very complete draining which the filter receives from time to time, it would seem that water other than the filter effluent was associated with the complications which were mentioned in the 1898 report of the Lawrence Water Board, and which were referred to in the report of the State Board of Health for 1898, on page 26.

The use of two grades of sand in a water filter is quite unusual, and is, of course, accounted for as a part of the method adopted for preventing excessively high rates of filtration through recently-scraped portions of the filtering surface. It is noted that the two kinds of sand are specified in Fig. 2 according to their effective size, as is the usual custom at present, and not with regard to the "maximum rate" of the sand, as was the case on the original drawings and in the earlier reproductions. By "maximum rate" is meant the quantity of water in millions of gallons per acre daily which a sand, when clean, will allow to flow through it when the acting head is equal to the thickness of the sand layer, and the temperature of the water is 50° Fahr. Numerous observations* have been made showing that the friction of a sand to the flow of water increases very materially as the sand continues in service. In fact, it is quite uncertain what coefficients for different conditions should be used for converting effective size into maximum rate. Much assistance in the future will be secured if more attention be paid than at present to the maximum rate which sands of a given effective size offer to the flow of water in a filter as they are continued in service.

Although the Lawrence filter is still unprovided with a cover, the advantages which would result from the operation of this filter during winter weather if it should be covered are so apparent, both from the standpoint of economy and of uniformity in the hygienic efficiency of the filter, that there is practically nothing to be said on the discussion of this point.

With regard to the method of intermittent operation of the Lawrence City filter, it was considered at the time it was designed that, at least during some portions of the year, the amount of atmospheric oxygen dissolved in the Merrimac River water might prove inadequate to facilitate to a desired degree the nitrification of the organic matter contained in the water and stored in the filter, unless supplemented by air admitted to the filter by draining the sand. Exhaustive studies at a later date showed that for this river water artificial aeration is not needed, because, practically without exception, the effluents are found to contain sufficient oxygen for nitrification processes.

* See the paper by Mr. H. W. Clark, in the 1894 Report of the Massachusetts State Board of Health, p. 706.

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In the investigations made elsewhere in this country it is found that the organic matter contained in river waters used for public supplies is of such a character as to stability that nitrification is now considered as a general proposition to be much less of a factor in water purification than it was ten years ago. Many river waters contain more organic matter than the Merrimac at Lawrence, but very few contain so much of an unstable nature such as is caused by very recent sewage pollution, and which is capable of being readily acted upon by the bacteria. Present evidence indicates that intermittent filters will never find extended use in water purification, on the ground that if a water contains so much unstable organic matter that the oxygen which it contains naturally, and also by aeration before application to the filter, is inadequate for normal bacterial actions, then such a water should not be considered favorably for a public supply.

The intermittent method of operation possesses two distinct disadvantages. One refers to the vertical channels produced in the sand layer, due to escaping air upon filling the filter after draining, and which has a tendency to lower the bacterial efficiency of the plant. The other refers to the air which remains entrained in the sand layer, and which increases the friction offered to the passage of the water. This latter point affects the cost of operation by increasing the amount of sand to be removed to relieve clogging. It also increases the cost by reducing the capacity of the plant below the normal (for continuous filters) after it has been in service for some years.

In the Lawrence City filter the effect of air in the pores of the sand has shown itself less in reducing the bacterial efficiency than it has in increasing the amount of sand to be scraped from the surface. The influence of the channels through which the water could pass without adequate purification is very likely disguised by other factors in this large plant. It certainly was well-defined in the 16-ft. experimental filter at the Experiment Station, as will be noted by comparing the records of Filters Nos. 3 *b* and 8 *a* in the Reports of the Massachusetts State Board of Health. These two filters were put in operation at about the same time as the city filter, and contain the same kind of sand. No. 3 *b* was operated intermittently, and No. 8 *a* continuously. A summary of the leading results from the operation of the city filter and of these two experimental filters, receiving practically the same water, up to January 1st, 1900, is shown in Table No. 31.

From this summary it is seen that the intermittent filters did not surpass the continuous filter in the removal of organic matter or of color, while they gave a somewhat less satisfactory bacterial removal, although the differences all come within about one-half of 1% of the numbers in the applied river water. It will further be noted that the intermittent experimental filter gave a distinctly smaller yield of filtered water between scrapings than did the experimental continuous filter; and that

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the intermittent city filter gave much smaller yields than the experimental filters, thus increasing materially the volume of sand to be removed per million gallons of effluent. Although the sand removed from the small filters is very likely less than would be the case with any municipal filter operating on this water, it is considerably in excess of the amounts removed in some European filters operating upon very clear water.

TABLE No. 31.—Summary of Leading Results of Operation of Water Filters at Lawrence, Mass., from September, 1893, up to January, 1900.

	City filter.	No. 3 *b*.	No. 8 *a*.
Kind	Intermittent.	Intermittent.	Continuous.
Area (acres)	2.44	0.005	0.005
Date of completion, 1893	Sept. 20.	Sept. 23.	Sept. 26.
Million gallons of water filtered	6 960	26.8	31.6
Average yield, in million gallons per acre daily	1.25	2.35	2.77
Total number of complete scrapings	79	86	79
Average number of days between scrapings	29	26	29
Average number of scrapings per year	12.6	13.8	12.6
Average yield per period between scrapings, in million gallons per acre	36	62	80
Estimated total amount of sand removed by scrapings	Thickness, in inches	59	42.5	42.0
	Cubic yards per acre	8 000	5 700	5 600
Estimated average thickness of sand removed by scraping, in inches	0.75	0.49	0.53
Average volume of sand removed, in cubic yards per million gallons of effluent	2.79	1.06	0.89
Average color in applied water	0.43	0.41	0.41
Ditto in effluents*	0.38	0.29	0.29
Percentage of color removed	12	29	29
Average nitrogen as albuminoid ammonia in applied water	0.199	0.086	0.186
Ditto in effluents	0.094	0.189	0.088
Percentage of albuminoid ammonia removed	53	52	53
Average number of bacteria per cubic centimeter in applied water	9 000	9 000	9 000
Ditto in effluents*	95	65	45
Percentage of bacteria removed	98.9	99.3	99.5

* Excluding results of first month of operation.

In the paper a statement is made as to the estimated average amount of suspended matter which the Merrimac River water contains, namely, 10 parts per million. This figure is approximately correct, but it does not give an adequate idea of the capacity of this river water to clog a sand filter, when compared with the turbidity of western and southern waters. It is the suspended organic matter, of a more or less gelatinous nature, which acts as an important agent in clogging the filters to which the Merrimac River water is applied. To record properly the clogging powers of a water, a method should be devised which should include in a correct manner not only the turbidity due to suspended silt and clay, but also that due to organic

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matter. Obviously, a small weight of the latter, forming as it does a gelatinous film at and near the sand surface, does as much clogging as a much larger weight of mineral matter which produces a more granular appearance upon the sand surface.

The clogging in the Lawrence City filter appears to be caused, to a large degree, by the air entrained in the sand. This is made plain in Table No. 32, in which the leading data of operation of the Lawrence filter, as given in the paper, are compared with the data from the Mount Vernon filters, as given in *Engineering News*, May 30th, 1901. As a matter of general interest, the data from the Albany filter, as given by Mr. Bailey in *Engineering News*, August 7th, 1900, are also included in the comparison. It is not intended that these Albany data should be regarded as representative of that plant, as doubtless Mr. Bailey will soon publish the detailed evidence to date. While in a number of instances these results are only approximations, it is believed that they are sufficiently correct for present purposes, and that the averages are free from serious error. In both the Lawrence and Mount Vernon filters the costs are exclusive of charges due to removal of ice and snow. Fixed capital charges and the cost of analytical supervision are not included in the case of any of the plants.

TABLE No. 32.—Comparison of Approximate Average Results and Costs of Operation of Water Filters.

Filter	Lawrence.	Mt Vernon.	Albany.
Area, acres	2.44	1.1	5.6
Period included in averages	1896-1900	1897-1900	1899-1900
Million gallons of water filtered	5 820	2 740	3 570
Yield in million gallons per acre, daily	1.3	1.7	2.6
Total number of complete scrapings	75.	61.	9.6
Number of days between scrapings	24.	24.	25.
Number of scrapings per year	15.	15.	15.
Million gallons of effluent per acre between scrapings	32.	41.	66.
Cost of scraping per acre	$25.80	$....	$17.50
Cost of wheeling out per acre	69.00	....	35.20
Sum of scraping and wheeling out	94.80	66.65	52.70
Thickness of sand layer removed per scraping, in inches	0.75	0.61	0.71
Cubic yards of sand removed per acre per scraping	100.	82.	95.
Cubic yards of sand removed per million gallons of effluent	3.14	2.00	1.44
Cost per cubic yard for scraping and wheeling out sand	$0.95	$0.82	$0.55
Ditto for washing sand	0.47	0.33	0.40
Ditto for replacing sand	0.28	0.36	0.43
Sum of cost per cubic yard of handling sand	1.70	1.51	1.38
Ditto per million gallons of effluent	5.36	3.03	1.98
Miscellaneous and general expenses per million gallons	0.56	0.11	0.19
Total cost of operation per million gallons of effluent, exclusive of ice	5.92	3.14	2.17

In regard to the relative costs of scraping the filters and of wheeling out the dirty sand, this information is not available for the Mt. Vernon filters. At the Lawrence filter it is to be recalled that

wheeling out the sand involves two handlings of the material, it being first wheeled to a temporary dump at the side of the filter. Mr. G. W. Fuller.

In studying these data it is seen that the cost per cubic yard of sand, for scraping, wheeling out, washing and replacing in the Lawrence filter is only about 10% in excess of what might be called a normal charge, notwithstanding that the total cost of operation, exclusive of ice, is more than double the sum found by European experience with American rates of wages, and as found in several places in this country. The chief difference is found in the large volume of sand removed per million gallons of effluent; and it seems to be a logical conclusion that this feature, due evidently in a large measure to clogging caused by entrained air, is the principal explanation of the high cost of operation of the Lawrence filter.

Although the cost of filtration at Lawrence is above the normal for this type of water, this question becomes a matter of relative insignificance when we consider the great work which the filter has accomplished in reducing typhoid fever in this city. During the past five years, excepting a few weeks following the special repairs in the underdrains, the annual death rate from this disease has been less than 20 per 100 000 inhabitants, a rate which for American cities is about as low as can be expected in cases where public water supplies are drawn from unpolluted sources. This record is the greatest testimonial which can be paid to the splendid results accomplished by the Lawrence City filter.

In their discussion of the typhoid statistics of Lawrence, the authors neglect to call especial attention to the low death rate which the city now enjoys, and confine their comments to the percentages of reduction of typhoid fever based on the records before and after the installation of the filter. These comparisons are of interest, but they have their limitations, as typhoid data seem to move more or less in cycles of uncertain lengths. While the authors are correct that had the filter not been erected at Lawrence the typhoid rate would possibly have been less during the past seven years than during the preceding period of that length, the reason is not to be found in a study of the general records of the State of Massachusetts, but in those of the Merrimac Valley above Lawrence. Notwithstanding the fact that the general pollution of this river is steadily increasing year by year, it is true that there has been a reduction in typhoid fever in those cities above Lawrence which sewer into this river or its tributaries. The reason for this is almost solely due to the marked reduction in typhoid fever at Lowell, following the introduction of a ground-water supply. In fact, in the remaining cities in the upper Merrimac Valley, as far as the speaker has been able to ascertain, the rate of increase of population has, on an average, more than offset such decreases as have occurred in the typhoid fever death rates.

Mr. G. W. Fuller.

Referring to the relationship which the filter has brought about between the removal of bacteria from the Merrimac River water and the removal of typhoid fever, it will be noted that, in round numbers, this filter, on an average, has effected a removal of about 99% of the bacteria in the applied water. In this connection the speaker desires to call attention to the fact that in 1891, when the typhoid fever rate in Lawrence was highest, there was a removal of from 90 to 95% of the river water bacteria as the water passed through the distributing reservoir and piping system of the water-works. In the early winter of 1898-99, following the repairs on the underdrains, and when a mild epidemic of typhoid fever appeared, the filter, on an average, effected a removal of about 93% of the river water bacteria. To those who are inclined to the opinion that in the case of sewage-polluted waters it is a matter of indifference whether or not a filter is carefully operated, the records from Lawrence will furnish much food for reflection.

Mr. Soper.

GEORGE A. SOPER, Assoc. M. Am. Soc. C. E.—This paper is accorded a warm welcome in the mind of the speaker, for two principal reasons. First, it is valued for its intrinsic merit; it is full of carefully collected facts, and these are set forth in a way which gives ample assurance that the authors desired to collate their information in an intelligent and impartial manner. The paper is also peculiarly acceptable to the speaker for the opportunity it affords him to make a few inquiries under circumstances which seem to insure their full answer.

Seeming Inconsistency Between the Filter and its Results.—A few years ago, when the speaker was engineer for one of the mechanical filter companies, it was considered part of his business to answer some of these inquiries regarding the Lawrence filter. The question usually originated with laymen, who, as delegates of municipalities, or representatives of water companies, were investigating the relative merits of slow and rapid filters, and who had passed through the hands of those who knew how to say a good word for slow filtration. They had heard a great deal about the excellent results at Lawrence, and were a little confused as to how those results were obtained. The speaker's principal interrogatory, therefore, is, what are the conditions upon which the efficiency of the Lawrence filter especially depends?

Basing a knowledge of filtration upon principles and precepts founded very largely upon the classic experiments made at Lawrence for the Massachusetts State Board of Health, it has sometimes been considered that in construction and operation the Lawrence filter is not one from which superior results could be expected. The facts at first laid down as necessary to its success, seem to have been disregarded in actual practice. Thus, for example, it was stated by the engineer of the works that one of the essential qualities of the design requires that the inlet water shall be shut off once a day, and the

pump kept running until the sand is uncovered and the water within the sand drawn out, so that the sand can fill with air from top to bottom. This was a condition made necessary by the principle of intermittent filtration. Again, it was intended to clean a portion of the bed every day, and this portion, when cleaned, was said to be incapable of conveying so large a quantity of water that it could not be properly purified in its passage, unless there were an extreme drawing down of the head, which would require close watching at the pumps. Mr. Soper.

In recent years it has been customary to operate the Lawrence filter in a manner which has been described by the authors as not intermittent, but practically continuous, and they state that it has made very little difference in the results.

It is learned from the paper that there has been a great deal of drawing down of the head, and it seems there has not always been a very close watching of the pumps. Yet chemical and bacterial evidence is not wanting to show that the quality of the effluent is not seriously affected by these seeming departures from original principles. The peculiar features of the plant are not restricted to the points named. In the Lawrence works we have a filter operating at 5 000 000 galls. per acre per day, instead of the 2 000 000 to 3 000 000 galls. generally accepted as the proper limit. We have a filter which is stopped and started every day instead of one which is run at a uniform rate for periods generally exceeding a month. The sand is never equally clean or dirty over the whole area; from which it would seem that, different parts of the bed being more or less clogged, and so offering unequal degrees of resistance to the flow of water, different rates of filtration and not a uniform rate would necessarily be obtained through the bed. No period of rest is given the clean filter for the formation of a coat of *schmutzdecke*, nor is the first of a new effluent thrown away as likely to be impure. After cleaning the bed, raw water is poured down on the filter until the sand is flooded; filtered water is not backed up under it. Then, the sand is not of uniform depth; the differences in depth are intended to be made up by differences in the size of the grains of sand, so that uniformity in filtration further depends on a nicety in adjusting the filtering material. These are all unusual features for a slow filter, and the peculiar success which seems to have attended the plant consequently calls for a few words of explanation.

How are the excellent results accounted for? Apparently, either on the basis that, in the past, the laws of filtration have been observed more narrowly than necessary, or that the Lawrence plant has elements of advantage which are not ordinarily taken into account. The speaker is aware that recent facts from the Lawrence Experiment Station throw light on some of the points referred to, and his experience with water filters assists in reconciling others, but it would be interesting to hear from some of the gentlemen who have been connected with the Law-

Mr. Soper. rence works, as to how, in general terms, they regard the apparent inconsistencies pointed out between the filter and its results.

Indications of Ground-Water in the Effluent.—Leaving the question as to how the principles elsewhere understood to control efficiency in slow filtration have bearing upon the operation of the Lawrence plant, the speaker desires to call attention to a feature which has not been touched upon by the authors, but which may have some influence in bringing about the excellent results attributed to the filter. It is desired to ask how far the accidental inflow of ground-water, or infiltration of river water to the bottom of the filter, may affect the results. The length of the filter excavation is 750 ft., with a width of 140 ft., and it lies within an initial distance of 50 ft. from the river. The bottom of the filter bed is understood to be unpaved, and open to the inflow of such water as may be present from 6 to 8 ft. below the level of low water in the river. The paper does not state the character of the excavated material, but it is stated that the old filter gallery which runs along the filter on the land side, was built in a stratum of coarse gravel. The gallery was 300 ft. long and was located 200 ft. from the river. It was tested in September, 1877, and found to be able to collect 275 000 galls. of water per day. When the filter was built the infiltration gallery was incorporated with it, and is now used as a conduit to carry the filter effluent to the pumps. The length of the infiltration conduit has been extended to a total length of 710 ft. in a direction previously regarded as one likely to develop a considerable amount of ground-water.

The question occurs here: How much of the Lawrence water is filtered, and how much is ground-water? The results of chemical analyses of applied Merrimac River water and effluent from the Lawrence filter for the years 1893 to 1899 have a bearing upon this point. The data are given by the authors in Table No. 17. Examining the determinations of those dissolved matters which, under ordinary circumstances, are little if at all affected by filtration, it is found that the river water is purer than the effluent. The figures which represent chlorine and hardness are remarkable; the average increase in these matters being 9.1% and 50%, respectively. The total solids are increased 33.6 per cent. If, from the data for total solids, is subtracted the average weight of suspended matter found in Merrimac water, the increase in the amount of dissolved solids which appear to result from filtration is 78.8 per cent. Even if a liberal allowance is made for the conversion of organic matter into mineral matter, which generally takes place in well-operated filters, there is still a heavy increase of solids to be accounted for. It is possible that the sand parts with some of its mineral matter to the water which passes through it, but many considerations render it unlikely that so great an increase in dissolved matters, and especially chlorine, could come

from this source. The conclusion seems irresistible that a considerable quantity of ground-water finds its way into the drainage system of the filter plant. Mr. Soper.

The application of the point is obvious. It means, unless explained, that the filter is credited with filtering at a greater rate and with, perhaps, a higher bacterial efficiency than is actually the case. The presence of an uncertain, though possibly small amount of ground-water, or infiltrated river water, would affect the accuracy of the reports of the filter's action in such a way that those who have the interests of the case most at heart must have carefully collected information along this line. If so, the speaker would like to have some of it.

The Reduction of Bacteria.—The results of bacteriological analyses of the water pumped at Lawrence, indicate that comparatively few bacteria pass through the filter bed. The proportion removed is given as 99.1% for the last year. It is noticed that there has been a steady decrease in the number of bacteria found in the effluent since the filter was built. From an average of 150 per cubic centimeter in the effluent in 1894, after the filter had been put in good running order, the number of bacteria has run down to an average of 55 per cubic centimeter for the year 1899. During the same time the raw water appears to have improved in quality also. In the six years mentioned, the number of bacteria in the Merrimac River water at Lawrence has fallen from an average of 10 800 to 5 800 per cubic centimeter. It would be interesting to know how these decreases are accounted for, and whether they are to be at all ascribed to improved methods of bacteriological technique, or represent solely an improvement in the quality of the water. The question has bearing upon the actual number of bacteria present in the water pumped from the filter. This is an important point in water purification, as it is of less significance in the average case that a filter shall remove 99% of the bacteria, than that it shall leave only 50 or fewer organisms to the cubic centimeter.

Reduction in the Typhoid Fever Death Rate.—A word on the subject of typhoid fever, which is the ultimate and most crucial phase of the question before us: The Lawrence filter was constructed in 1893 to protect the people of Lawrence against typhoid fever, which was considered to be communicated to them largely through the medium of the public water supply. The death rate from typhoid fever at Lawrence for the six years prior to the construction of the plant averaged 12 per 10 000 of the population. For the six years following construction, the death rate from typhoid per 10 000 was 2.6. The difference is 78 per cent.

The authors very properly point out that this percentage of reduction is not to be ascribed solely to the operation of the filter plant.

Mr. Soper. Other factors combined to reduce the mortality. Lest there should be some misconception, the authors have indicated the nature of some of these influences. Thus, they state that one favorable factor lay in improved sanitary conditions, while another was better methods of medical treatment. To the speaker's mind, a very important factor lay in the educational effect produced by the great amount of attention which was called to the danger of drinking impure water at about this time. Letters were sent by the Massachusetts Board of Health to the Mayor of the city, notices were posted in the mills and public places, and there was a general campaign against the disease which had for some time been epidemic in various cities along the Merrimac River. As an evidence of the popular response to the warning against polluted water it is said that large quantities of bottled water were sold, and that many people boiled the city water before drinking it. Just how efficacious this educational factor may properly be considered to have been, it is impossible to say, but its effect in all probability, was very considerable.

It has been frequently stated that fluctuations in the number of deaths from typhoid fever at Lawrence followed similar changes at Lowell before the Lawrence City filter was built. A relation of cause and effect seems to have been firmly established in this matter by those who have carefully investigated the facts. Has this relation continued to the present time? Data relating to monthly variations in the typhoid fever death rates for the two cities are not at present accessible, but if the available statistics for the period from 1893 to 1899 are examined, a plotting of the number of deaths from typhoid fever each year together with the similar data from Lawrence will show a general correspondence in the curves. That is to say, there was a very notable decrease in each case at about the time the Lawrence filter was built. From 53 deaths at Lowell in 1893, there was a sharp decline to 18 in 1897; 24 in 1898, and 17 in 1899. Here there is an influence highly favorable to a reduction of the death rate at Lawrence which has apparently been overlooked. Using the last two factors referred to as conditions which contributed to a reduction in the typhoid fever death rate at Lawrence, it can hardly be denied that the number of typhoid fever victims would have been greatly decreased by the filter.

This leads to the question: What actual saving in life has resulted from the construction of the filtration plant? The authors consider, after deducting the effect of various influences and tendencies, that the filter has resulted in the prevention of the death of 40 out of every 100 persons who would otherwise have died of typhoid fever in Lawrence. That is to say, in a city of 62 520 inhabitants, the lives of 8 persons were saved from death last year. It is not the speaker's intention to ask that the figure given in this estimate be qualified or reduced. The question is not one that is susceptible of proof, but rests

properly as a matter of opinion. The saving mentioned, or half this saving, would amply justify the installation of a filter, but it is desired to ask of the gentlemen who have made a study of the Lawrence plant, how, in the face of so many seeming imperfections in construction and operation, such good results can be accounted for? Mr. Soper.

There is little doubt that these inquiries cover ground which is familiar to those who are intimately acquainted with the details of construction and operation of the plant under discussion, and that they are prepared to give full and complete answers to all questions which may be asked. It is hoped that they will regard the present inquiries as worthy of consideration, and that they will give the profession the benefit of the light which they, better than any others, are able to throw upon these topics.

GEORGE C. WHIPPLE, Assoc. M. Am. Soc. C. E. (by letter).—This paper, upon the operation of the Lawrence filter, is most interesting and complete, yet there is one subject to which the authors have not alluded, namely, the microscopic organisms and their growth upon the sand and in the filtered water when stored in the reservoir. The probable reason for the omission is that, unlike the experience of many cities with open filters, Lawrence has had no trouble from these organisms. Mr. Whipple.

TABLE No. 33.—NUMBER OF MICROSCOPIC ORGANISMS IN THE WATER OF THE MERRIMAC RIVER, AT LAWRENCE, MASS.

	1890.	1891.	1892.	1893.	1894.	1895.	1896.
January	....	3	....	7	15	14	3
February	11	55	16	38	20	45	5
March	....	....	5	53	56	20	12
April	16	9	109	19	49	46	12
May	8	101	153	93	126	434	145
June	45	119	247	289	461	1 690	225
July	310	320	160	346	165	1 163	356
August	113	678	82	175	469	708	142
September	19	79	88	113	139	108	58
October	71	43	58	88	83	85	12
November	5	34	29	10	25	40	6
December	7	19	11	15	4	2	47
Average	61	133	87	104	134	363	86

List of prominent organisms.	Number of times in which the organisms exceeded 50 per cubic centimeter.
Synedra	21
Protococcus (chlorococcus)	7
Fragilaria	2
Crenothrix	4
Tabellaria	2
Chroöcoccus	1
Selenastrum	1
Dinobryon	1

Mr. Whipple. As a rule, river waters do not contain large numbers of algæ, protozoa, etc., and the Merrimac River, at Lawrence, is no exception to this rule. The records of the Massachusetts State Board of Health show that, with the exception of *Synedra* and *Protococcus*, no organism has ever been present in numbers greater than 500 per cubic centimeter. Table No. 33, compiled from these records, gives the number of organisms during each month from February, 1890, to December, 1896, together with a list of the genera which have been observed in numbers greater than 50 per cubic centimeter, and the number of occasions on which this figure was exceeded.

TABLE No. 34.—NUMBER OF ORGANISMS, EXPRESSED IN STANDARD UNITS PER SQUARE CENTIMETER, UPON THE SURFACE OF CERTAIN FILTERS.

FILTER.	LAWRENCE CITY FILTER.	MOUNT VERNON, N. Y.	PITTSBURG, BED NO. 1.	POUGHKEEPSIE.
Date of Examination.	April 12, 1900.	April, 1900.	May 4, 1900.	April 10, 1900.
Diatomaceæ—				
Cyclotella	4 000	20 000	108 000	14 000
Gomphonema	4 000			
Melosira	32 000	80 000	12 000	
Navicula	4 000	30 000	6 000	21 000
Synedra	24 000	160 000	1 092 000	35 000
Tabellaria	4 000		12 000	
Asterionella		700 000		
Cymbella		10 000		
Fragilaria		50 000		
Eunotia				7 000
Surirella				7 000
Coscinodiscus				7 000
Ceratoneis				7 000
Meridion				14 000
Chlorophyceæ—				
Closterium		10 000		
Cosmarium		10 000		
Conferva				14 000
Cyanaphyceæ—				
Cœlosphærium		150 000		
Oscillaria				14 000
Schizomycetes—				
Crenothrix		20 000	90 000	
Protozoa—				
Cyphoderia	4 000	10 000		
Ciliata		10 000		
Trachelomonas		10 000		
Vorticella		10 000		
Anthophysa				7 000
Rotifera—				
Amiræa		10 000		
Rotifer	4 000	10 000		
Other organisms—				
Nais	4 000			
Anguillula		10 000		
Chætonotus		10 000		
Total organisms	84 000	1 320 000	1 320 000	147 000

Not only are there comparatively few organisms in the applied water, but few organisms seem to grow upon the sand or in the water above the sand. With the filter operated intermittently, heavy growths

Mr. Whipple.

of algæ upon the sand would not be expected, as the drying of the sand surface would not be favorable to them. Recently, however, the filter has been operated continuously, thus giving opportunity for such growths to occur, but, even now, the *schmutzdecke* contains comparatively few organisms. During the year 1900, through the kindness of Mr. A. H. Salisbury and Mr. M. F. Collins, the writer had the opportunity of examining samples of scum from the Lawrence filter at frequent intervals. All the samples were noted for their paucity of microscopic organisms. The largest number was found on June 29th, 1900, after the bed had been in use 34 days. On that date there were 200 000 organisms per square centimeter of sand surface. The smallest number was 16 000 per square centimeter, on May 8th, after the bed had been in use 25 days. These figures practically represent the organisms strained from the water, rather than a growth on the sand, and are so low that they are far from being the controlling factor in determining when a bed must be scraped. It is probable that they affect, but little, the cost of scraping. In order to give an idea of the number of organisms that may be found upon the surface of a filter, Table No. 34 has been prepared from the examination of scum samples made during the year 1900.

TABLE No. 35.—NUMBER OF MICROSCOPIC ORGANISMS IN THE WATER OF THE DISTRIBUTING RESERVOIR AT LAWRENCE.

	1890.	1891.	1892.	1893.	1894.	1895.	1896.
January	9	10	11	20	0	23	0
February	7	9	23	4	2	24	904
March	4	7	24	2	78	4	1 386
April	27	24	69	55	85	604	5
May	7	12	66	74	14	85	0
June	19	10	12	40	1	57	44
July	202	196	74	135	214	136	156
August	46	56	70	91	189	27	8
September	57	26	52	147	570	1 717	22
October	90	143	12	10	464	2	120
November	60	47	31	12	7	21	20
December	37	33	5	1	4	0	10
Average	47	48	37	49	135	225	223

List of prominent organisms.	Number of times in which the organisms exceeded 50 per cubic centimeter.
Crenothrix (leptothrox)	6
Protococcus (chlorococcus)	12
Synedra	2
Cyclotella	1
Botryococcus	1
Anabœna	1

At times the numbers rise to 5 000 000 and even 10 000 000 per square centimeter. Even in covered filters, as at Albany, N. Y.,

Mr. Whipple. and West Superior, Wis., numbers of more than 1 000 000 are sometimes obtained. At West Superior *Crenothrix* forms the bulk of the growth.

Apparently, there has been no trouble from growths of microscopic organisms in the filtered water when stored in the distributing reservoir. This reservoir is 694 ft. long, 375 ft. wide and 25 ft. deep, and has a capacity of 40 000 000 galls., or about 14 days' supply. Yet, with this comparatively long storage, the numbers of organisms in the reservoir water have been singularly low. Table No. 35, compiled from the published reports of the Massachusetts State Board of Health, shows that no extensive growths have occurred.

No results have been published since 1896, and that fact may be taken as an indication that no trouble has been caused by odor-producing organisms. Since the installation of the filter there has been an improvement in the odor of the water in the distributing reservoir The "mouldy" and "faint, musty" odors have, to a large extent, disappeared, and, at the present time, the water often has no odor.

It should be observed, however, that since the installation of the filter in 1893, the organisms have shown a slight tendency to increase (this is especially true in the case of *Crenothrix*), and, even if the figures are still too low to be of practical importance, it is to be hoped that in the interests of Science, the State Board of Health will continue to make these observations.

In reference to the bacteria in the filtered water, the authors mention the fact that the numbers in the distributing reservoir are often higher than those in the effluent after passing through the filter, or in the service taps in the city. They suggest as a reason that the increase may be due to bacteria entering the open reservoir from the air, or to the development of bacteria in the water. In the writer's opinion, these causes do not account for the large numbers often noticed, and he is inclined to the belief that the oftentimes excessively high numbers may be attributed to local contamination of the reservoir samples. The reports of the State Board of Health state that these samples are collected from a tap in the effluent gate-house. It may easily happen that, with a pipe used as little as this one undoubtedly is, abnormal results may be obtained. If this be the case, the bacterial results of the reservoir water are of little value, and the authors are justified in omitting the results from their table.

The general decrease in the number of bacteria in the pipes of the distributing system, and the occasional increase during the summer correspond with what has been observed elsewhere. Apparently, this summer increase is due to the growth and decay of organisms in the pipes.

Mr. Tribus. LOUIS L. TRIBUS, M. Am. Soc. C. E. (by letter).—Every few years the world takes up a new fad: sometimes useful; sometimes foolish. Some push it to the extreme, but, in time, the good in it becomes accepted and it reaches its true position in life's economy.

In sanitary science the ruling fad of the present is filtration, and it possesses so much of inherent merit as almost to justify the furor; still we do not need to run wild over it, or consider it as the universal panacea, as is a tendency to-day.

The past two decades have indeed marked wonderful advancement in the subject. At the beginning of that period, with a few exceptions, filtration meant only a straining out of suspended matters; nothing was thought of any organic purification.

Many were the filter tanks and hollow porous walls filled with sand, gravel or charcoal, used to strain out the vegetable growths and silt; becoming in themselves, because usually very improperly cared for, the breeding places for organisms more harmful than all the matters removed.

Occasionally, however, some systems did approximate to the more accepted forms of the present; the Poughkeepsie plant, designed by Mr. Kirkwood, being noteworthy; though its bacterial action was not considered at all as a factor, until probably after the Lawrence plant was inaugurated.

The chemist and the biologist have contributed a great deal of information as to Nature's method of action, and the public at large has even adopted and uses glibly, the terms "bacterial life," "nitrifying organisms," "colonies of germs," etc., as if it thoroughly understood the whole subject.

Many systems of treatment have been taken up, especially abroad, but they all gradually crystallize into the two, "slow" sand and "rapid" sand with chemical assistance; while results and efficiencies have been claimed at times scarcely within the realm of reason.

Still, great actual progress has been made, and greater intelligence is being daily brought to bear in the application of the principles concerned.

Percentages are dangerous forms of comparison, yet there seems to be no better method at times. In speaking of the removal of bacteria it may often be very misleading. For instance: 99 would seem to be a high figure, yet 99% removal from water carrying 5 000 bacteria per cubic centimeter gives no better result than 95% from water carrying 1 000 per cubic centimeter; in each case 50 per cubic centimeter remaining in the effluent.

The writer has no better method to propose, but simply calls attention to the danger of blindly specifying percentages, as is frequently done. Each case needs to be treated according to its own

Mr. Tribus. special local conditions and not by any general rule, which is very often particularly inapplicable.

The secret of success of any filter is not its percentage of bacterial removal, but, through careful operation, not alone careful design, the removal of all dangerous germs; this removal does not mean elimination of all bacteria, for many are entirely harmless to the human system, but the intelligent propagation of those which are harmless that shall aid in the destruction of those which are harmful.

It certainly is not taking too advanced a stand to say that every water that requires filtration to free it from polluting matters should be filtered under the constant care and daily examination of one trained in biological research as well as common sense.

In the discussion of this paper the decreased death list from typhoid is shown clearly, both at Lawrence and elsewhere, consequent on such installations, but, to demonstrate the point, we need not give all the credit to the engineer and the biologist.

When the fact became thoroughly demonstrated that typhoid was a filth-germ disease, cities began to cleanse the districts from which it was likely to emanate, and so lessened materially the numbers of cases. Then a far better comprehension of the disease itself brought more efficient treatment by physicians, thus cutting down the proportion of deaths to cases.

The same causes that induce a community to seek a purer water supply, if it be needed, are the ones that are effective from the sanitary and medical standpoints as well. The result attained is to be applauded, and filtration is justly entitled to a goodly share of the credit in the places where it has been established.

The financial value of the factor "saving of lives," because of improved municipal conditions, is not often appreciated.

Taking at Lawrence the average total deaths from typhoid for six years before and six years after the construction of the filter plant, as given in the paper, of 53 and 14 per year, respectively; cutting down the 53 average by one-third, to represent better medical and sanitary conditions, to say 35, compared with the 14, shows a saving of 21 lives per year at least, creditable to filtration of the supply; allowing $150 per year, as a low estimate of net gain to the community for each inhabitant; multiplying by the 21 equals $3 150, the net saving in the first year. In twenty years, a reasonable period to take for productive human life, the aggregate net earnings of the 21 persons saved each year, would, without any interest being added, equal $660 000, or an average of $33 000 per year that could be spent for interest, sinking fund and maintenance of the filter plant, without exceeding the real financial value of the service to the community; as against say $15 000 or $16 000 for the same items actually spent at Lawrence.

The 420 persons saved from death would also make an interesting exhibit. Mr. Tribus.

Did city officials, or the people as a whole, more generally understand and realize these facts, sanitation and purification would receive much more attention than has been or is even now being given to the case.

While endorsing, without any qualification, filtration of water supplies, whenever polluted, it would seem that there can be no question that, if a source of supply free from contaminating influences can be secured, such source is to be preferred in comparison even with the best methods of artificial purification possible; conditions of constructive and operative costs, permanence and accessibility for inspection being equal.

Bearing on that statement, in 1895 the State Board of Health of Massachusetts, reporting on a new supply for the Metropolitan District, after estimating that a filtered supply from the Merrimac River could be secured for less money (including operations, capitalized), than any other source considered, stated, that though it had been shown by the works at Lawrence (built from Mr. Mills' plans and described in the paper under discussion), that "waters as polluted as those of the Merrimac can be effectually filtered and rendered safe for domestic use," yet its conclusion was that "in the opinion of the Board it will be better to pay 10% more for a supply from a source that has not been polluted." Such a source was supposed to have been found.

It is not always possible for our cities to secure such uncontaminated supplies, even by those having large capital, and therefore ability to go great distances; hence, the tremendous importance of filtration and its possibilities, and the great value of such conscientious investigation and pioneer construction, as described in this paper.

John H. Gregory, Assoc. M. Am. Soc. C. E. (by letter).—The authors are to be congratulated on presenting such a complete history of the construction and operation of the filter which has contributed so greatly to the advancement of water purification in this country. Various questions relating to the design and operation of the filter have been too ably discussed by others for the writer to add anything on these points, but a comparison of the costs of operation of the Lawrence filter with other filters may not be amiss. Such figures as are readily available have been compiled and are well worthy of careful study by those interested in the operation side of the question. Mr. Gregory.

In one of the tables presented by the authors, the total costs charged to various items are given. In order to reduce these to a basis for comparison with other plants, the figures have been worked over, using as a unit the cost per million gallons. The results are given in Table No. 36.

Mr. Gregory.

TABLE No. 36.—ITEMIZED STATEMENT OF COST OF OPERATION OF THE LAWRENCE FILTER, PER MILLION GALLONS, 1895–1900.

Items.	1895.	1896.	1897.	1898.	1899.	1900.
Scraping	$2.45	$1.23	$1.73	$1.60	$1.76	$1.76
Scraping and sanding		1.33	0.74	0.25	1.08	1.03
Conveying	1.27	1.07	1.94	1.37	1.26	1.16
Washing	0.96	1.92	1.72	1.32	1.24	1.25
Removal of snow and ice	2.68	2.14	2.06	2.87	1.97	1.93
General	0.43	0.45	0.75	0.59	0.43	0.60
Total	$7.79	$8.14	$8.96	$8.00	$7.74	$7.73
Total, minus snow and ice	5.11	6.00	6.90	5.13	5.77	5.80
Pumpage, million gallons	1 040	1 100	1 130	1 210	1 170	1 210

The average total cost of filtration for the six-year period was $8.05 per million gallons. The average cost of removing snow and ice for the same period was $2.27, and deducting this from the total cost leaves $5.78 as the average cost per million gallons, minus snow and ice.

In Table No. 36, the costs of superintendence and bills are charged to the various items. In order to separate these items, and thus obtain the cost of labor for the various operations, the authors have kindly furnished the writer with the figures for 1900, from which the following and other tables have been deduced.

TABLE No. 37.—ITEMIZED STATEMENT OF COST OF OPERATION OF THE LAWRENCE FILTER, PER MILLION GALLONS, FOR 1900.

Item.	Cost.	Percentage of cost.
Scraping (labor)	$1.50	19
Scraping and sanding (labor)	0.90	12
Conveying (labor)	1.02	13
Washing (labor)	0.94	12
Removal of snow and ice (labor)	1.56	20
General (labor)	0.38	5
Superintendence	0.91	12
Bills	0.52	7
Total	$7.73	100
Pumpage, million gallons	1 210	

In a paper recently read before the American Water-Works Association, the cost of operation of the Albany filters from July 26th, 1899, to December 29th, 1900, was given by Mr. George I. Bailey. The figures are presented in Table No. 38. In this table the cost of superintendence and supplies is charged to the various items, but the cost of maintaining the laboratory is not included.

TABLE No. 38.—ITEMIZED STATEMENT OF COST OF OPERATION OF THE ALBANY FILTERS, PER MILLION GALLONS, 1899–1900, 501 DAYS. Mr. Gregory.

Item.	Cost.	Percentage of cost.
Scraping	$0.24	14
Wheeling out	0.45	27
Refilling	0.32	20
Incidentals	0.13	8
Lost time	0.10	6
Washing	0.42	25
Total	$1.66	100
Amount filtered, million gallons	6 463	

In order to make a comparison with the Lawrence figures for 1900, Table No. 37, in which the cost of labor is given, Mr. Bailey has kindly furnished the writer with figures from which Table No. 39 has been deduced.

TABLE No. 39.—ITEMIZED STATEMENT OF COST OF OPERATION OF THE ALBANY FILTERS, PER MILLION GALLONS, 1899–1900, 501 DAYS.

Item.	Cost.	Percentage of cost.
Scraping (labor)	$0.16	10
Wheeling out (labor)	0.30	18
Refilling (labor)	0.22	13
Washing (labor)	0.27	16
Incidentals (labor)	0.09	6
Lost time (labor)	0.07	4
Superintendence	0.33	20
Supplies	0.18	11
Wash water	0.04	2
Total	$1.66	100
Amount filtered, million gallons	6 463	

In comparing the labor costs at Lawrence and Albany, it should be stated that laborers are paid $2 for a day of 8 hours at Lawrence, and $1.50 for the same time at Albany.

The oldest filter plant in the United States, from which reliable figures are available, is at Poughkeepsie, N. Y. In Table No. 40 is given the cost of operating this plant from 1877 to 1900, a period of 24 years. These figures have been compiled from the annual reports of the Board of Water Commissioners of Poughkeepsie. The writer has endeavored, as far as possible, from the information given in the reports, to separate such costs as should be charged to construction, and to include in the table only those relating to operation. This is

Mr. Gregory.

TABLE No. 40.—ITEMIZED STATEMENT OF COST OF OPERATION OF

Items.	1877.	1878.	1879.	1880.	1881.	1882.	1883.	1884.	1885.	1886.	1887.
New sand	$0.13	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Cleaning	.....	.....	$0.98	$0.68	$0.75	$0.64	$1.17	$0.80	$0.45	$1.37	$1.23[1]
Washing	1.46	.....	1.82	1.91	1.65	1.83	2.14	1.28	1.01	1.81[2]	
Replacing	0.65	.....	.....	.....	.....	.....	.....	.....	.....	1.16	.....
Removing		.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Removing ice	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Repairs	.....	.....	.....	.....	0.06	0.03	0.27	0.14	0.01	1.58[3]	.....
Operation (so-called)	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Total	$2.24	$2.85	$2.80	$2.59	$2.46	$2.50	$3.58	$2.22	$1.47	$5.92	$1.23
Total, minus snow and ice	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Care of grounds	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Pumpage, million gallons	569	529	536	513	557	508	509	496	591	634	645

[1] Includes cost of removing ice.
[2] Old sand all removed and washed.
[3] Includes so-called cost of operation.
[4] Includes cost of construction of bridges.

stated here, inasmuch as the writer's figures may not agree exactly with those which others may have deduced. For example, Mr. Charles E. Fowler, Superintendent of the Poughkeepsie Water-Works, stated, in a paper read before the New England Water-Works Association in 1898, that the average cost of maintenance and operation, for a period of 20 years from 1877 to 1896, inclusive, was $2.99 per million gallons. The figure as deduced by the writer for the same period would be $2.92 per million gallons, the difference between the two being due, as stated above, to the charging of certain costs to construction.

The average total cost of operation at Poughkeepsie for the period covered by Table No. 40, 1877 to 1900, inclusive, has been $3.27 per million gallons. During the last three years only, 1898 to 1900, inclusive, has the cost of removing snow and ice been reported separately. This cost has averaged $0.47 per million gallons, and deducting this from the total average cost, $2.42, for the same period, leaves $1.95 as the cost per million gallons minus snow and ice.

In a recent number of *Engineering News* was published the cost of operating the filters at Mt. Vernon, N. Y. These figures have been reduced to the basis of cost per million gallons, and are given in Table No. 41.

The average total cost of filtration for the six years, 1895 to 1900, inclusive, has been $3.16 per million gallons. For the last five years, 1896 to 1900, inclusive, the average total cost has been $3.33 per million gallons, and deducting from this the average cost of removing snow and ice for the same period, $0.35, leaves $2.98 as the average cost per million gallons, minus snow and ice.

THE POUGHKEEPSIE FILTERS, PER MILLION GALLONS, 1877–1900. Mr. Gregory.

1888.	1889.	1890.	1891.	1892.	1893.	1894.	1895.	1896.[6]	1897.	1898.	1899.	1900.
.....	.....	.....	.....	$1.21		.....	.....	.....	$5.63	.....	.....	.....
1.33[1]	1.76[1]	.....	1.40[3]	1.81	3.46	2.50[3]	$2.88[3]	$1.05	0.67	$1.19	$1.08	$1.31
.....	.....	.....	.....	.....	0.93	0.37	0.46	.....	.....	0.85	0.56	0.82
.....	.....	1.86	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	0.48	0.36	0.60
0.14	0.31	0.38	0.04	0.40[5]	0.14	0.11	0.22	.....	6.42[6]	.....	.....	0.12[9]
.....	.....	1.76	.....	.....	.....	.....	.....	3.22	.....	.....	.....	.....
$1.47	$2.07	$4.00	$1.44	$3.42	$4.53	$2.98	$3.56	$4.27	$12.72	$2.52	$1.95	$2.85
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	$2.04	$1.59	$2.25
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	$1.55	$1.34	$1.47
609	572	603	628	696	667	634[7]	687[7]	664[7]	615	611	717	653

[5] Includes cost of new sand washer.
[6] Repairs to old bed and inlet basin, and removal and wasting of old sand.
[7] Exclusive of leakage.
[8] Cost of tools only.
[9] New bed built, and put in operation December 17th, 1896.

TABLE No. 41.—ITEMIZED STATEMENT OF COST OF OPERATION OF MT. VERNON FILTERS, PER MILLION GALLONS, 1895–1900.

Items.	1895.	1896.	1897.	1898.	1899.	1900.
Cleaning and renewing beds	$1.99	$1.90	.....	.....	.....	$1.32
Cleaning beds	.....	.....	$1.35	$1.85	.....	.....
Overhauling sand bed	.....	.....	.....	.....	.....	1.43
Cost of new sand	.....	.....	.....	.....	.....	0.24
Washing sand	.....	.....	0.50	0.50	.....	0.75
Placing sand on beds	.....	.....	0.52	0.49	.....	.....
Removing ice	.....	0.03	0.21	0.42	$0.81*	0.21
Miscellaneous	0.02	0.06	0.05	0.04	.....	0.10
Total	$2.01	$1.99	$2.63	$3.30	$4.45	$4.05
Total, minus removing ice	.....	1.96	2.42	2.88	3.64	3.84
Consumption—Million gallons	493	608	646	686	726	679

* Only one item reported.

For comparing conveniently the average cost of operating the four plants, Table No. 42 has been compiled.

Passing now from a study of the cost of operation to a study of the work in detail, some very interesting figures relating to the Albany and Lawrence plants can be deduced. The figures from which the deductions of the Albany plant have been made, are taken from Mr. Bailey's paper, and the writer, being well acquainted with the conditions at Albany, has made an analysis of these figures, the results of which are given in Tables Nos. 43 to 46.

Mr. Gregory. TABLE No. 42.—Comparison of Cost per Million Gallons of Operation of Filters at Lawrence, Albany, Poughkeepsie and Mt Vernon.

	Lawrence	Albany.	Poughkeepsie.		Mt. Vernon.	
	1895–1900. 6 years.	1899–1900. 501 days.	1877–1900. 24 years.	1898–1900. 3 years.	1895–1900. 6 years.	1896–1900. 5 years.
Average total cost	$3.05	$1.66	$3.27	$2.42	$3.16	$3.33
Average cost of removing snow and ice	2.27	.....	.....	0.47	.....	0.35
Average cost, minus snow and ice	5.78	.....	.....	1.95	.....	2.98

TABLE No. 43.—Detailed Statement of Scraping of the Albany Filters, 1899–1900, 501 Days.

	Hours of labor.	Cubic yards.	Barrows.	Cost of labor.
Total	5 481	7 942	100 145	$1 047.35
Per bed (0.7 acre, approx.)	43.2	62.5	789	8.25
" acre	61.4	*89.0	1 121	11.73
" 1 000 sq. ft.	†1.41	2.04	25.8	0.27
" cubic yard	0.69	.....	12.6	0.13
" hour of labor	.....	1.45	18.3	.....
" 8 hours of labor	.....	11.6	146	.....
" million gallons filtered	.....	1.23	155	0.16

* Average thickness removed, 0.66 in. = $\frac{1}{18}$ in. = 0.055 ft. approximately.
† 1.41 hours per 1 000 sq. ft. = 710 sq. ft. per hour = 11.8 sq. ft. per minute = 5 680 sq. ft. per 8 hours.

Scraping includes the removal of covers from the ventilator shafts in the vaulting, and the scraping into piles of the dirty sand from the surface of the bed.

TABLE No. 44.—Detailed Statement of Wheeling out at the Albany Filters, 1899–1900, 501 Days.

	Hours of labor.	Cubic yards.	Barrows.	Cost of labor.
Total	10 238	7 942	100 145	$1 956.88
Per bed (0.7 acre, approx.)	80.6	62.5	789	15.41
" acre	115	89.0	1 121	21.92
" 1 000 sq. ft.	2.64	2.04	25.8	0.50
" cubic yard	1.29	.....	12.6	0.25
" hour of labor	.....	0.77	9.8	.....
" 8 hours of labor	.....	6.2	78	.....
" million gallons filtered	.....	1.23	15.5	0.30

Mr. Gregory

Wheeling out includes the handling of the running plank, shoveling the scraped material into wheel-barrows and removing it to the court, raking and screeding the bed, and replacing the covers on the ventilator shafts. The average round trip distance for removing the sand is about 500 ft.

During the last five months that the writer was connected with the construction of the Albany plant, the filters were put in operation one after another, as soon as completed, and the opportunity was afforded of keeping a force account of all the operations. Mr. Bailey's figures cover all those obtained by the writer, but in those of Mr. Bailey the cost of raking and screeding the surface of the sand just prior to refilling and starting a run is included in the cost of wheeling out. For a portion of the time the writer kept these figures separately, and, as they are of interest, they are given in Table No. 45.

TABLE No. 45.—Detailed Statement of Raking and Screeding the Albany Filters, 1899.

Hours of labor per bed (0.7 acre, approx.)	22.6
" " " " acre	32.2
" " " " 1 000 sq. ft	9.74
Square feet per minute of labor	22 5
" " " hour of labor	1 352
" " " 8 hours of labor	10 810
Cost of labor per bed	$4.25
" " " " acre	6.05
" " " " 1 000 sq. ft	0.14

In Tables Nos. 43 and 44 the details of scraping and wheeling out at the Albany filters have been given separately. In order to compare the relative rates of work at Albany and Lawrence, these two tables have been combined in one, Table No. 46, covering what may be called cleaning the filters, and including all the operations mentioned under the heads of scraping and wheeling out.

TABLE No. 46.—Detailed Statement of Cleaning (Scraping and Wheeling out) of the Albany Filters, 1899–1900, 501 Days.

	Hours of labor.	Cubic yards.	Barrows.	Cost of labor.
Total	15 719	7 942	100 145	$3 004.23
Per bed (0.7 acre, approx.)	124	62.5	789	23.66
" acre	176	89.0	1 121	33.65
" 1 000 sq. ft	*4.05	2.04	25.8	0.77
" cubic yard	1.98	.....	12.6	0.38
" hour of labor	.....	0.50	6.4	.....
" 8 hours of labor	.....	4.0	51	.....
" million gallons filtered	.....	1.23	15.5	0.46

* 4.05 hours per 1 000 sq. ft. = 247 sq. ft. per hour = 4.1 sq. ft. per minute = 1 980 sq. ft. per 8 hours.

Mr. Gregory. The process of scraping at Lawrence includes, as the authors state, the scraping up of the dirty sand, its removal to the bank by wheel-barrows, and the raking and screeding of the bed. It will be seen that this compares very closely with what the writer has called cleaning at Albany, as the distance the dirty sand is wheeled is about the same at each place. The figures for 1900 at Lawrence are given in Table No. 47.

TABLE No. 47.—Detailed Statement of Scraping of the Lawrence Filter, 1900.

	Hours of labor.	Cubic yards.	Cost of labor.
Total	8 296	2 350	$1 830.00
Per acre	228	*64.7	50.08
" 1 000 sq. ft.	†5.25	1.49	1.15
" cubic yard	3.53	.....	0.77
" hour of labor	.....	0.28	.....
" 8 hours of labor	.....	2.26	.....
" million gallons filtered	.....	1.94	1.50

*Average thickness removed, 0.48 in. = 1/25 in. = 0.04 ft.
†5.25 hours per 1 000 sq. ft. = 191 sq. ft. per hour = 3.2 sq. ft. per minute = 1 524 sq. ft. per 8 hours.

TABLE No. 48.—Comparative Statement of Cleaning Filters at Lawrence, Albany and Poughkeepsie.

	Lawrence.	Albany.	Poughkeepsie.	
	1900.	1899-1900.	Without bridges.	With bridges.
Hours of labor per acre	228	176	290	181
" " " " 1 000 sq. ft.	5.25	4.05	6.66	4.17
" " " " cubic yard	3.53	1.98	.....	.....
Square feet per hour of labor	191	247	150	240
" " " minute of labor	3.2	4.1	2.5	4
" " " 8 hours of labor	1 524	1 980	1 200	1 920
Cost of labor per acre	$50.08	$33.65	.....	.....
" " " 1 000 sq. ft.	1.15	0.77	.....	.....
" " " " cubic yard	0.77	0.38	.....	.....
" " " " million gallons	1.50	0.46	.....	.....

At Poughkeepsie, prior to 1886, and again since 1897, the filters have been cleaned by the same method as that used at Lawrence and Albany. Mr. Fowler stated that the labor required under normal conditions, was one man for each 150 sq. ft. of surface cleaned per hour, including wheeling and elevating to the bank. In 1886 two bridges were constructed, spanning the two sections of the bed. These bridges were movable, and, from them, platforms were suspended a little above the surface of the sand. The men stood upon these platforms and scraped up the dirty sand, throwing it on the plat-

forms. When a portion of the bed had been cleaned, the bridges were moved to one end of the bed and the sand was thrown up on the bank. By this process, under normal conditions, one man would clean 240 sq. ft. of surface per hour. The cost of labor at Poughkeepsie was not given, but these figures, with others derived from them, are compared in Table No. 48 with corresponding ones at Lawrence and Albany. Mr. Gregory.

In Tables Nos. 49 and 50 will be found the details of the sand-washing operations at Lawrence and Albany.

TABLE No. 49.—Detailed Statement of Washing Sand at the Lawrence Filter in 1900.

	Hours of labor.	Cubic yards.	Cost of labor.
Total	5 152	3 000	$1 130.00
Per cubic yard	1.72	.....	0.38
" hour of labor	.....	0.58	.....
" 8 hours of labor	.....	4.66	.....
" million gallons filtered	.....	.....	0.94
* Wash water per cubic yard of sand	.....	.....	.....

* Wash water, 10 volumes of water per volume of sand = 270 cu. ft., or 2 020 galls. of water per cubic yard of sand.

TABLE No. 50.—Detailed Statement of Washing Sand at the Albany Filters, 1899-1900, 501 Days.

	Hours of labor.	Cubic yards.	Barrows.	Cost of labor.
Total	8 923	5 673	83 299	$1 706.25
Per cubic yard	1.57	.....	14.7	0.30
" hour of labor	.....	0.64	9.3	.....
" 8 hours of labor	.....	5.1	75	.....
" million gallons filtered	.....	.....	.....	0.27
* Wash water per cubic yard of sand	.....	.....	.....	0.04

* Wash water, 12½ volumes of water per volume of sand = 337.5 cu. ft. or 2 525 galls. of water per cubic yard of sand.

At Lawrence, washing includes the screening of the dirty sand at the first or traveling hopper of the washer and the transportation from the draining boxes to the permanent storage bank, together with such attention as the washer itself requires.

At Albany, washing includes the handling of scraped sand from the storage piles to the sand washer, attendance on the sand washer and removing the washed sand to a storage pile.

At both places the ejector type of machine is used, the nozzles at Lawrence being horizontal, while those at Albany are vertical.

Mr. Gregory. The processes of sanding at Lawrence, and of refilling sand at Albany are nearly the same. The authors state that sanding includes removing the clean sand from the storage bank to the surface of the bed, distributing it in layers and smoothing the surface, also the labor necessary to prepare the surface of the bed for this operation. At Albany, refilling includes removal of washed sand from storage piles into the filter, loosening the top layer of the sand about 6 ins. deep and leveling the new sand.

The work at both places would be directly comparable but for the fact that the Lawrence figures include one scraping of the surface just prior to sanding. As the labor for this is but a small portion of the total charged to sanding, the writer believes that the figures for Lawrence in 1900, given in Table No. 51, are fairly comparable with those for Albany given in Table No. 52.

TABLE No. 51.—Detailed Statement of Scraping and Sand of the Lawrence Filter in 1900.

	Hours of labor.	Cubic yards.	Cost of labor.
Total	5 000	3 400	$1 090.00
Per cubic yard	1.47	.....	0.32
" hour of labor	.....	0.68	.....
" 8 hours of labor	.....	5.44	.....
" million gallons filtered	.....	.....	0.90

TABLE No. 52.—Detailed Statement of Refilling Sand at the Albany Filters, 1899–1900, 501 Days.

	Hours of labor.	Cubic yards.	Barrows.	Cost of labor.
Total	7 437	5 673	89 081	$1 421.15
Per cubic yard	1.31	.....	15.7	0.25
" hour of labor	.....	0.76	12.0	.....
" 8 hours of labor	.....	6.1	96	.....
" million gallons filtered	.....	.....	.....	0.22

Although the cost of operation at Lawrence has been high, it should be remembered that the filter is an uncovered one, that it is in a climate subject at times to severely cold weather, and that it has been operated under rather disadvantageous conditions. But in spite of all this the money has been well spent, and the good work accomplished by it has been recognized by engineers, and is fast being recognized by the public at large.

Mr. Fowler. CHARLES E. FOWLER, Esq.* (by letter).—In the writer's experience with the open filters at Poughkeepsie, covering a period of twenty years, he has not observed that the delivery of the filters, immediately after scraping, was in any wise affected by air entrained in the sand, although it was the invariable practice, until within a few years, to fill the filters by flowing the water over the sand.

The manipulation of the filter prior to and during scraping, however, will affect the immediate delivery very materially. In fact, the writer has known a scraping to be performed in such manner and under such conditions as to absolutely stop the flow, and the operation had to be repeated, in different manner and under different conditions, before any water could be coaxed through. Undue pressure or head, habitually or frequently carried, has a strong tendency to produce the result mentioned. The practice of forking or deep raking also tends in the same direction. With these prior conditions and the bed a little too moist it will require but a small amount of inattention on the part of the person in charge, and improper handling of the shovels by the scrapers, to effect a very material reduction in the delivery immediately after scraping. If the bed is in good order, the immediate delivery may be varied by the depth of sand removed in scraping, even when the filling is from below.

The maximum head allowed on the Poughkeepsie beds, during recent years, is 4 ft. Sometimes they are scraped when the head is less, depending upon the rate of increase of head and the condition of the applied water. Ordinarily, the head immediately after scraping is less than 1 ft.

It became necessary to scrape the beds in April, last, when the applied water was in an unusually turbid condition. The head before scraping was 3.2 ft. Intentionally, less than the usual depth of sand was removed, and the head on starting up was 2.0 ft., the beds being filled from below. The chief object in filling these beds from below is to avoid furrowing the sand by the inflowing water.

In scraping the Poughkeepsie beds, the sand is not first placed in heaps, as described and illustrated in the paper, but, when lifted from the surface by the shovels, it is deposited in barrows. Sandals are not used. In smoothing the surface of the sand, after scraping, the back of a rake only is used. Even the shortest teeth are not used, the endeavor being to cause as little disturbance of the sand surface as possible. Shovels are not allowed to be stuck vertically in the sand.

The minimum cost of scraping is about $46 per acre, or 0.12 cent per square foot. The men employed are picked up for the occasion, and are such as do not readily find employment elsewhere. They are about the lowest grade of laborers. The minimum number employed is 23, and they occupy 2 days, of 8 hours each, in scraping $1\frac{1}{3}$ acres.

* Superintendent and Engineer of Public Works, Poughkeepsie, N. Y.

Mr. Fowler. Each man receives $1.50 per day. The walls of the basins are vertical, and the top of the coping is about 6 ft. above the sand surface. The cost of scraping given above includes depositing in barrows, wheeling to the corners of basins, throwing up to top of coping and trimming back on pile. Two laborers are employed about the beds and grounds, but are not used in scraping. During the intervals between scrapings these men wheel to the storage pile the sand thrown up from the filters.

It was formerly the practice to wash the scraped sand and store it for replacing; the washing being done by 2 to 4 men through the spring, summer and fall months. It is the present practice to store the sand unwashed, during the year, and when the time for replacing arrives, usually in October, the sand is washed and replaced on the beds as washed. A much greater number of men is employed, and the operation requires about 16 days to wash and replace one year's scrapings on an area of $1\frac{1}{3}$ acres. The sand is thrown into a hopper at the storage pile and forced by a water-jet to a series of inclined troughs discharging into a tank from which the silt and dirty water overflows, and a water-jet at the bottom forces the sand through a pipe to a tank set over one edge of a bed, the water flowing off at the top and the sand discharging in a pile on the surface of the bed.

The sand is distributed over the bed by barrows. There is no carting, and the only wheeling is on the bed. In 1900 the cost of labor for washing sand was 32.2 cents per cubic yard. The cost of labor for washing and replacing was 58.4 cents per cubic yard. The total quantity washed and replaced was 910 cu. yds. The average quantity per hour was 6.78 cu. yds., and per day, 54.3 cu. yds. All the quantities given are for clean sand measured on the bed. The pay of the laborers was $1.50 per day of 8 hours.

The yearly cost of ice removal varies with the thickness of the ice, the number of times taken off, distance to be removed, etc. For the three years prior to 1900–01 the cost varied from $146.61 to $440.49; average, $281.01. In 1900–01 the ice was taken off only once, and the cost was $613. The ice was much thicker than usual, averaging $13\frac{1}{2}$ ins. on the old bed and 18 ins. on the new bed, each bed being $\frac{2}{3}$ acre in area. The distance to which the greater portion of the ice was removed was also materially greater. The work was done entirely by manual labor, and occupied 62 hours. The minimum number of men employed was 54 and the maximum 64. The rate of pay was $1.50 for 8 hours. The men comprised all kinds of laborers who chanced to be in want of employment.

The ice was sawed in one direction and broken by chisels in the other. The cakes were floated to a run at the side of the basin, and pulled up by men with pikes. The surface of the water was maintained as near the coping as practicable, generally less than 1 ft below

the top. After the cakes were lifted out they were pushed on nearly horizontal runs to the place of deposit. This is the most costly part of the operation. The cost of cutting and lifting the ice out of the basins was $289.32 for the entire area, or $192.88 per acre. The total cost per acre of filtering area was $408.66. The total cost per million gallons filtered was 94 cents. There is no removal of snow other than with the ice. The ice covering the beds is not allowed to touch the sand at any time if it can be avoided. Mr. Fowler.

WILLIAM B. FULLER, M. Am. Soc. C. E. (by letter).—Thanks are due to the authors for their painstaking labor in compiling and making available these data concerning the Lawrence filter; for it is by a study of such data from existing plants that we can hope to improve the efficiency of and decrease the expenditures on future works of this kind. Mr. W. B. Fuller.

In applying the experience gained at Lawrence to other places, it must be borne in mind, however, that the conditions there are not such as are likely to occur at many other localities; two factors, especially, contributing to this result, namely, the extreme pollution of the river and the small amount of turbidity. Both of these factors are very favorable to high efficiency in slow sand filtration, and undoubtedly account for the favorable showing made by the plant, which in so many particulars contains features which are now avoided.

From the writer's point of view, the great value of this paper lies in the presentation of the detailed costs of operation and the consequent bringing out, in discussion, of the comparative costs of operation at other works. It is shown that a large percentage of this cost is due to the removal of the dirty sand from the beds, its transportation outside, washing, storing and transportation back again, and a study of these costs shows two matters which should receive serious attention in the design of new plants:

First: The keeping of the amount of sand handled per million gallons at its lowest point consistent with efficiency, and

Second: The reduction in cost of handling the sand by the substitution of machinery for hand labor.

In relation to the reduction in thickness of the sand layer to be removed, Mr. George W. Fuller suggests the clogging caused by entrained air as a serious factor affecting the quantity of sand handled, and Mr. Collins suggests, among others, rain, snow, ice, freezing of the exposed sand, and unskilled labor. The troubles due to entrained air are now generally recognized and avoided by the use of methods of refilling from below and by the use of positive heads during operation. Most of the troubles suggested by Mr. Collins can be avoided by covering the filter beds, thus adding new reasons to the already long list in favor of covered filters.

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In relation to the reduction of cost in handling the sand it appears that the methods now in vogue are costly and cumbersome; they are methods copied from the operation of the older plants of England and Germany, scraping by hand labor, transportation by wheel-barrows or carts, and washing and storing involving much hand labor, but it would seem that such methods are hardly adapted to this country with its higher-priced labor.

The space set apart for the storage of sand is also an important element in the first cost of the plant, and one which it would be desirable to eliminate.

For several years the writer has been of the opinion that the principle of water transportation, greatly extending the idea described by the authors for the transportation of sand to the sand-washer, will materially reduce such costs.

He would suggest the use of many stationary hoppers situated in different portions of a filter bed with the necessary piping leading to a central sand washer, and a transportation pipe from the last sand washer to the place of storage.

He would also suggest that the units of a filter plant be made of such small size as would permit of two units being out of commission at one time. The dirty sand would then be removed from one unit, transported to the sand washer, washed and transported to place in the other unit without the intervention of unsightly storage heaps or costly storage courts, and with a minimum of hand labor. Further, it would be advisable to use covered filters well lighted by artificial light to allow of continuous work in stormy, dark or freezing weather, thus insuring the employment of a permanent trained force, working under uniform conditions, during the entire year, a result which, in itself, as pointed out by Mr. Collins and Mr. Bailey, would insure a maximum of efficiency in such labor.

A point worth noting in relation to the removal of ice at Lawrence is, that owing to the large size of the unit the ice could be floated from one side to another without removal for several scrapings. If the filter plant had been divided into smaller units, this could not have been done, and the resultant cost of removing ice would have been greater.

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MORRIS KNOWLES, Assoc. M. Am. Soc. C. E., and CHARLES G. HYDE, C. E. (by letter).—The writers are in no small measure gratified by the response with which the publication of their paper on the Lawrence, Mass., city filter has been met. They feel that the nature and scope of the discussions have been such that the paper has been given a value which it would otherwise have lacked. This is due not only to the extended reference to the general principles involved in filtration, and to the salient features of the design and operation of the particular filter in question, and of other works, but also to the careful

and elaborate statements and tabulations of cost which appear in several discussions. Messrs. Knowles and Hyde.

In the sections which follow, the writers have made such additional or explanatory statements as will serve to complete the information relative to the filter, where the discussions have shown the data in the original paper to be meager; and have suggested answers to the important questions which have been presented.

Water-Shed Populations.

A recomputation of the total population resident in 1900 upon the water-shed of the Merrimac River above Lawrence has been made upon a different basis from that used in producing the figure originally presented by the writers. The revised estimate of 480 000 persons agrees very closely with that of Mr. Hazen on page 309. The figure for urban population deduced by the writers, however, differs from that of Mr. Hazen on account of the assumption, in the first case, of 5 000 as a limit of rural population in a township, while Mr. Hazen's estimate is based upon a limiting rural population of 4 000.

The writers are glad to have the correct estimate of the total water-shed population of the Hudson River above Albany, and have included the changed figure in Table No. 3. But they would, at the same time, call attention to the fact that if the rural population of 1890 be added to the urban of 1900,* a total of 89 per square mile is obtained. This means, as compared with Mr. Hazen's new figures, an apparent reduction of 30% in rural population in ten years: in fact, however, the decrease is not as great; for the upper limit of rural population, in the table used as a basis by the writers, is 1 000, while, for the statement on page 312, Mr. Hazen has used 4 000 as the limit.

Construction.

The suggestions of Mr. Collins, in that part of his discussion entitled, "Conditions Due to the Construction of the Filter in One Unit," relative to the erection of division walls across the filter, appear to be most pertinent. If this feature were introduced, there is little doubt that the work of maintenance would be rendered more satisfactory in many ways, and much more economical.

Investigations in Regard to Clogging, and Work of Renewal, of Underdrains.

Since the presentation of the paper, the writers have had opportunity to finish the compilation of the necessary data, and to prepare a comprehensive statement regarding the conditions which led to the repair work during the years 1898 and 1900, together with some refer-

* *Transactions*, Am. Soc. C. E., Vol. xliii, p. 284.

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ence to the results obtained. Allusions to these matters have already been made in several discussions on the paper, and the information contained in these and in the following statements will, it is believed, answer Mr. Soper's inquiries and Mr. Weston's question.

Conditions for a Few Years after Construction of Filter.—For two years following the construction of the Lawrence filters, no serious difficulty was found, for any continued interval of time, in obtaining from it a supply of water entirely adequate for the demands of the city, without the necessity of operating the pumps for more than about two-thirds of the day, nor during Sundays and holidays, and without producing an excessive loss of head. Especially during the warmer portions of the year, the filter was found to work admirably, although the average loss of head under ordinary working conditions was always, after a while, greater than in the beginning.

The circumstances of operation and of scraping during the coldest weather, however, were unsatisfactory, owing to the lack of a cover over the filter. On the water surface thick layers of ice would form, which, when the filter was drained for scraping, lowered with the water down to the sand, thus disturbing the *schmutzdecke*. When the water rose again on the filter, portions of the sand were brought up with the ice, leaving the surface raw and broken, sometimes to a considerable depth. In addition, the freezing of the sand surface wherever it was exposed for scraping, the necessity for scraping quickly (which, unfortunately, led to the practice of scraping only those beds nearest the pumping station, and of allowing those further away to remain uncleaned for long periods during the winter), together with the lower temperature of the water, all combined to gradually increase the loss of head. The serious effects produced by this state of affairs obtained throughout all seasons, but especially during the cold weather, to such an extent that, to force the required amount of water through the filter, the water in the pump-well was drawn down to a level below the point of discharge of the underdrains of the beds, thus allowing air to pass through them back into the filter

Later, the filter appeared to be clogging elsewhere than at the surface, until, to some extent during the cold weather of 1895–96, and to a greater degree during the winter of 1896–97, it was found that the usual quantity of water could not be obtained from the filter, even with the maximum possible loss of head; consequently, the rate was diminished and the hours of pumping increased. During the winter of 1897–98, these conditions became greatly aggravated, and, in February of the latter year, the pumps were run day and night, with interruptions only to allow the water to rise in the pump-well to a sufficient height to provide water for the pumps. The loss of head was constantly equal to the maximum difference in elevation between the water on the surface and the bottom of the underdrains, while

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the surface of the water in the conduit was often much lower than the latter. Notwithstanding the continuous pumping, which oftentimes could not be carried on at a speed greater than two-thirds of the customary rate, the water in the reservoir, the capacity of which was equivalent to about two weeks' supply for the city, continued to lower, until half the storage was exhausted, and a partial water famine became imminent. Scraping the beds (which in the very coldest weather was almost impossible) appeared to produce but little benefit, and the conditions were such as to lead to grave apprehension on the part of the Water Board. The advent of warmer weather in March, 1898, however, permitted filtration to be carried on under somewhat more favorable circumstances, and the reservoir was gradually filled during the following month.

TABLE No. 53.—EXPERIMENTS TO DETERMINE THE VERTICAL DISTRIBUTION OF THE LOSS OF HEAD, INDICATING LOCATION OF CLOGGING IN LAWRENCE FILTER.

Depth of bottom of tube below surface of sand.	Position of sand layer relative to surface.	FEBRUARY 26TH, 1898.			MARCH 17TH, 18TH, 1898.		
		Percentage of Loss of Head.					
		Bed No. 5.	Bed No. 11.	Bed No. 21.	Bed No. 5.	Bed No. 11.	Bed No. 21.
8 ins.....	0 in.— 8 ins.	6	7	10	16	16	12
18 "	8 " —18 "	94	14	10	84	13	15
36 "	18 " —36 "	0	43	10	0	28	2
52 "	36 " to top of underdrains.	0	36	70	0	32	18
In gravel.....	Underdrains.				0	11	53
		100	100	100	100	100	100

Investigations of the Lawrence Water Board.—The serious aspect of the problem confronting the Water Board, of which, one of the writers, Mr. Knowles, was a member, led to the undertaking of a series of experiments to ascertain the relative extent of the clogging in different portions of the filter area, and the head required to force water through successive layers of the main body of the sand of the filter at these places. For this purpose there were used tubes of various lengths, having an interior diameter of about 3 ins. These were constructed of galvanized sheet-iron with conical bottoms, which were perforated with small holes and provided with a fine screen of brass wire at the junction of the base of the cone with the tube, in

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order to prevent the entrance of sand.* The tubes were set into the sand, over the southerly ends of the pipe underdrains, of Beds Nos. 5, 11 and 21, with the tops somewhat above the elevation of the surface of the water on the filter.

When the filter was in operation and the pumps were running, observations of the relative elevation of the surface of the water in the tubes and on the filter indicated the loss of head at various depths. On February 26th, a series of measurements was made with tubes placed in each bed to depths of 8, 18 and 36 ins. below the surface of the sand, and, on March 17th and 18th, the experiment was repeated with the addition of a fourth tube, the bottom of which rested directly on the top of the gravel underdrains, at an average depth of 52 ins. below the sand surface. On February 26th the observations were taken half-hourly, through an interval of 3 hours, and, on March 17th and 18th, taken hourly, through intervals of 7 and 8 hours, the fluctuation in the elevation of the water of the tubes being inconsiderable at all times. The results of these experiments are shown in Table No. 53.

During the experiments there was constantly maintained a total loss of head of 7 ft., which is the maximum obtainable with the surface of the water on the filter at its usual elevation. It will be seen that the loss of head in the upper sand layer was apparently, in all cases, a comparatively small percentage of the total, but it is possible that these results were vitiated by the fact that the tubes, being set into the sand to a depth of 8 ins. only, allowed the water on the filter, under a head of somewhat more than 3 ft., to force itself downward along their sides and up through the perforated conical bottoms, thus giving an apparent loss of head much less than the actual.

In Bed No. 5, near the pumping station, the entire loss of head was found to occur, during both series of observations, within the upper 18 ins. of the sand layer. In Bed No. 11, 21 to 29% of the total head was lost in the upper 18 ins., and the entire head was used up in passing through the main body of sand, the loss in the underdrains being inappreciable. In Bed No. 21, representing the easterly portion of the filter, farthest from the pumping station, where subsequent excavation proved the underdrain material to be badly clogged, the experiments showed that the loss of head in the upper 18 ins. of the main sand body was from 20 to 27% of the total, and that in the lower 16 ins. and underdrains was about 70%, of which the more comprehensive experiments of March 17th and 18th indicated that 53% was lost in the gravel underdrains alone.

The results of these studies indicated that there was a portion of the filter which contributed but little to the water supply for the city.

* Similarly constructed tubes have been used by the writers with gratifying success upon other work, and there is no doubt of their value as instruments to ascertain the variations in rates of filtration in different parts, and the loss of head through different depths of a filter.

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It was thought best, however, before doing anything further, to consult the State Board of Health, under whose advice the filter was built.

Application of Lawrence Water Board to Massachusetts State Board of Health for Advice; and the Reply.—At a meeting of the Lawrence Water Board, held March 4th, 1898, it was voted to address the following letter to the State Board of Health:

"The Lawrence Water Board respectfully represents that the municipal filter does not furnish sufficient water for the demands upon our system. We are led to believe, from limited experiments showing the loss of head, that portions of the filter are clogged.

"We desire to be advised by your Board in regard to the proper method to be adopted to restore the filter to its proper capacity, also to co-operate with us and advise what further experiments may be necessary to determine what portions need to be renewed."

In response to this request, the engineers of the State Board of Health made, during the following summer, an examination of the condition of the filter, and an investigation of the methods of manipulation and maintenance since its construction. By means of properly distributed tubes, sunk to various depths in the sand of many of the beds, the rate of draining of these beds was ascertained when the water was below the surface. Current meter observations were made of the rate of flow from the underdrain of each of the beds discharging directly into the old filter gallery and of the total flow from those discharging into the conduit. In several beds in the easterly portions of the filter, where tube measurements indicated a subsurface clogging, excavation was made to examine the lower sand layers and the condition of the underdrain material, principally in the vicinity of the underdrain pipes and the main conduit.

As a result of these studies, on September 19th, 1898, the State Board of Health sent the following reply to the Lawrence Water Board, in regard to the general condition of the filter and the method by which it might be improved:

"The State Board of Health received from you, on March 5th, 1898, the following request for advice with reference to improving the capacity of the Lawrence City Filter. * * *

"The Board has examined the available data as to the operation and management of the filter and the consumption of water as furnished from the records kept by your Board, and has made investigations to determine the quality and condition of the sand in the filter, and the condition of the underdrains as compared with their condition when first placed in the filter five years ago. The Board has also made many experiments to determine the cause of the reduced capacity of the filter and the best method of relief.

"Analyses of many samples of sand collected from all parts of the filter show that, while the effective size of the sand is probably slightly finer than when first placed in the filter, on account of silt carried by the river water, the increase in the fineness of the sand is not sufficient to cause a very material reduction in the quantity of water that would pass through the filter.

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"A careful examination of the height of water in the sand of the filter while it was being drained showed that, while the portion nearest the pumping station, amounting to approximately half the area, for the most part drained down rapidly, the water in the easterly half of the filter did not drain down readily, and in some portions of it remained, after several hours of draining, but little below the surface of the sand.

"Excavations were made through the whole depth of the sand and into the gravel forming and surrounding the underdrains. The sand was found to be clean and unobstructed throughout its full depth, and in the portion of the filter near the pumping station the gravel under the sand was found where examined to be in good condition, and the water appeared to drain freely through it. More attention was given to the easterly portion of the filter, where, as has been stated, the water did not drain readily from it. Here, as elsewhere, the sand was found in nearly all parts of it to be clean and unobstructed down to the gravel; but within the gravel and between the stones surrounding the drain pipes and in the joints of the drain pipes was found a deposit of gelatinous iron rust, together with some *Crenothrix*, a plant which grows in water containing iron, which deposit allowed very little water to enter the drain pipes. This deposit was more dense in the middle and upper portion of the layer of gravel, where air has entered this layer from the underdrains when the water in the pump-well has been drawn below the top of the underdrains.

"It is evident from the character of this deposit that air is essential to its formation; and the only way to prevent its formation in the future is to run the filter, as it was intended to be run from the time of construction, with the water in the pump-well always above the top of the underdrain, or not below the level of 31 ft. on the scale of heights.

"This portion of the filter can be rendered efficient by excavating trenches 10 ft. wide across the filter, in line of and over the underdrains, removing the gravel for this width, washing and relaying the pipes, and at the same time extending the lines of 4-in. pipes a distance of about 50 ft. from their present termination and placing clean gravel about them, as it was originally placed, adjacent to such pipes; then filling the trench above the gravel to a depth of 2 ft. with the washed sand which you have upon the bank, and filling above this and over the present top of the filter between the nearest carriers with the remaining sand from the excavation. This process should be continued over so much of the filter as does not allow the water to drain from the sand freely, which includes a large part of the easterly half of the filter and a small portion of the remainder.

"When the filter was constructed, you were advised that the whole surface should be cleaned once a month, and that certain portions should be cleaned oftener in times of freshets. Your records show that the easterly portion of the filter has been allowed to remain uncleaned in the cold season a much longer time, even reaching a period of four or five months, while the portion nearer the pumping station has been cleaned more frequently. To this fact is probably due the larger deposit of iron in the gravel of the easterly portion, and the Board regards it important that the original instruction in regard to cleaning the filter should be carried out to maintain its efficiency. To accomplish this, it is, in the opinion of the Board, necessary that the filter be covered, to effectually prevent the formation of ice upon its surface.

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"Upon completing the relaying of the underdrains and upon restoring the filter bed to its original depth, it should hereafter be maintained at that depth; and, to keep it in the most efficient condition for purifying the water, it should be run intermittently, as originally designed.

"You have found the filter delivering an abundant supply of water during the past four months, but from this you cannot conclude that it will continue to do so, for during the season of the formation of ice the sand will allow only about two-thirds as much water to pass through it as in summer. Your experience of last winter shows that it is necessary to act promptly.

"By entering upon the work of covering the filter with a roof to prevent freezing and of removing the gravel clogged with iron without delay, it is probable that enough work can be done before the beginning of winter to insure a sufficient supply of filtered water for the city."

On October 1st, the Lawrence Water Board submitted the foregoing communication to the City Council, with the request that a sum of money, not exceeding $37 000, be appropriated for the construction of a cover over the filter. At the same time the Council was informed that it was the intention of the Board to proceed at once with the work of reconstruction of the underdrains in accordance with the recommendations of the State Board of Health.

Renewals in 1898.—The work during this year consisted of the excavation of several beds in the easterly portion of the filter, which investigation had shown to be most clogged, the relaying of the pipe underdrains and underdrain material, and the replacing of old and new sand to the original grade of the surface; together with the screening and washing of various sizes of gravel and sand, in amounts sufficient for the work. All of this work was under the direct charge of Mr. Walter E. Spear, C. E., from whose report to the Water Board the writers have compiled many of the following data. Screening and washing the gravel and coarse sand was begun during the latter part of September, and was carried on until December 14th, whenever the weather permitted, except during the period of actual reconstruction of the filter beds.

Owing to the decreased rate of operation of the filter, due to its clogged condition, and the consequent diminished storage in the distributing reservoir, it became a difficult matter to obtain intervals of sufficient duration, with suitable weather, in which to carry on the actual work upon the filter. The first work of reconstruction was performed from October 18th to 21st, when the underdrain of Bed No. 13, and 16 ft. of that in Bed No. 18 were relaid. A severe storm prevented further operations, and it was not until about the middle of November that another suitable opportunity was secured. At this time, from November 14th to 18th, the underdrains of Beds Nos. 14, 15 and 19, and the remainder of No. 18 were relaid, and, together with Beds Nos. 12

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and 13, were brought to the required grade with sand. The next opportunity for labor on the filter came on December 6th, from which time work was pushed day and night to December 9th, and Beds Nos. 16, 17, 21 and 22 were improved and brought to grade.

The work of reconstruction was performed in accordance with the general recommendations of the State Board of Health. In each bed, with the exception of the first, a braced and sheeted trench 10 ft. in width was excavated, extending from the down-stream or northerly end of the underdrains, to a point near the southerly extremity of the beds, 15 ft. from the main distributing channel. The clean sand was thrown out of the trenches to each side upon the surface of the filter beds. The gravel underdrain material, which was found to be almost completely obstructed with a deposit of gelatinous iron rust, and a growth of *Crenothrix*, occurring especially in the upper layers, was wholly removed, together with the pipe underdrains.

TABLE No. 54.—Data of Underdrain Material used in Renewals in 1898.

Size of material.	Width of trench covered by material.	Depth of layers.
Inches.	Feet.	Inches.
2	3	4
1½	5	3
¾	7	1½
⅜	9	¾
3/16	9	¾
Coarse sand.	10	1
Total depth.....	.....	11

The bed of the filter, consisting of a mixed clayey and sandy material, was prepared for the relaying of the underdrains by placing over it a layer of unscreened gravel 1 in. or more in thickness. The old pipe underdrains were washed and relaid, and the lines extended in a southerly direction with 4-in. pipes, for a distance of about 50 ft. The grade of all pipes was 1 in 100, except in Beds Nos. 16, 17, 21 and 22, where the last 30 to 40 ft. were laid flat, in order to keep their elevation as low as possible. The original method of laying the pipes with open joints, so that the spigot of one approached but did not enter the bell of the next, was adhered to in reconstruction; each joint being surrounded with a 4 in. layer of 2-in. gravel. On the first gravel layer there were placed successive courses of material of the width and depth shown in Table No. 54.

A saving in the amount of 2-in. gravel surrounding the pipe underdrains was effected in Beds Nos. 16, 17, 21 and 22, by using this material at the joints for a distance of 6 ins. only, in either direction; the intermediate 12 ins. of each pipe being surrounded by 1½-in. gravel.

Upon the coarse sand, superposed on the 3/16-in. gravel, there were

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deposited from 2 to 2½ ft. of fresh washed sand, and upon this the sand which was excavated from the trench was placed, bringing each bed to the original elevation as designed; or about 1 ft. higher than the grade prior to reconstruction. In Beds Nos. 21 and 22, the flow of ground-water, which had originally caused some trouble in the construction of this portion of the filter, was encountered, and necessitated particular care in bracing and sheeting, especially in the northerly portion of the beds, in order to prevent the sand from working into the trenches.

The total cost of this work, as estimated by Mr. Walter E. Spear, Engineer-in-Charge, was $5 709.58, itemized as in Table No. 55:

TABLE No. 55.—Cost of Reconstruction in 1898.

Cubic Yards.	Items.	Cost.	Per Yard.
	Material.		
90	Unscreened gravel (used over bottom)..	$60.50	$0.67
50	Screened " (from P. Holland)...	103.95	2.08
421	Gravel of several sizes (screened at city bank).....	1 770.37	4.21
451	Gravel, washed.....	1 066.38	2.36
60	Coarse sand, screened and washed.....	35.11	0.59
....	Vitrified pipe, lumber and supplies	240.96	....
	Total for material.....	$3 277.27	
	Labor.		
1 600	Excavating and refilling trenches.....$1 008.19		$0.63
300	Removal of clogged gravel from under-drains.....	450.65	1.50
314	New gravel and drains.....	412.17	1.31
900	Washed sand in trenches.....	561.30	0.62
	Total for labor.....$2 432.31		
	Total for labor and material.....		$5 709.58
	Estimated Deduction for Unused Material.		
157	Screened gravel, several sizes.....	$553.90	
137	Washed gravel " "	323.93	
8	Coarse sand.....	4.68	$882.51
	Net actual cost of reconstruction in 1898.		$4 827.07
	Cost per bed.....	$536.43	

Results from Renewals in 1898.—An increase in the capacity of the filter became apparent immediately after the reconstruction of the first five beds; namely, Nos. 13, 14, 15, 18 and 19. As determined by the engineer in charge of the work at that time, basing an estimate upon the average rates of pumping for one day immediately before and one day after reconstruction, with conditions of loss of head and temperature reduced to 5.0 ft., and 50° Fahr., respectively, the gain in quantity received from the filter was about 20 per cent. By the same method of comparison, the apparent gain in the capacity of the filter, after the completion of the nine beds, using the results of October 16th, prior

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to the beginning of the work, and of December 14th, after its completion, was about 45 per cent.

According to an estimate made by the writers, reducing to a uniform basis the average data of the loss of head, temperature of water, rate of pumping, and actual quantities pumped for equal periods of 7 days, the apparent increase in the capacity of the filter was 33 per cent. The days used were October 12th to 18th, prior to the beginning of the work, and December 10th to 16th, after its completion; the surface condition of the filter being very nearly the same in each case, although probably slightly more favorable during the first period. A similar estimate, based upon the comparison of the first 7-day period, prior to reconstruction (October 12th to 18th), with a 4-day period immediately following a complete scraping of the surface of the filter, on April 5th and 6th, 1899, about 3½ months after the completion of the work, failed to show any increase in the capacity of the filter. However, it is likely that these later results are not wholly trustworthy, owing to the probable rapid filling of the filter after scraping, and the consequent imprisonment of air in the sand, thus producing a loss of head in excess of that which might be consistently expected with such surface conditions as obtained during the April period.

Renewals in 1900.—During the year 1899 no further work in improvement of the filter was attempted. In 1900, however, the work was resumed, four beds being rebuilt, completing the reconstruction of thirteen beds, representing the entire easterly portion of the filter. The work was carried on during the year under the direction of Mr. M. F. Collins, recently in immediate charge of the manipulation of the filter, and now Superintendent of Water-Works.

Beds Nos. 20, 23, 24 and 25 were reconstructed upon the following dates: No. 25 on May 25th; No. 24 on May 31st and June 1st; No. 23 on June 7th and 8th, and No. 20 on September 26th. The general methods of excavation and relaying used during the work of 1898 were followed during 1900, with the exception that the main conduit was cleaned and relaid, and, like the underdrain pipes, was placed upon cypress planks 2 x 8 ins. and 14 ft. long, fastened together endwise with cleats.

Table No. 56 shows the length of underdrain and conduit rebuilt, and the cost of the work directly upon each bed.

The condition of the underdrain material in all the beds rebuilt during this year was about the same as found in 1898, the gravel being greatly discolored, and choked with gelatinous iron rust, in the immediate neighborhood of the main conduit and pipe underdrains, but less clogged toward the up-stream ends of the beds, where the gravel was found to be in a fair condition. The drains and main conduits were so thoroughly choked in Beds Nos. 23, 24 and 25 that no appreciable amount of water would flow away into the main conduit at the bottom of the excavation, and pumping was required throughout the whole work.

TABLE No. 56.—AMOUNT AND COST OF WORK IN 1900.

Messrs. Knowles and Hyde.

No. of bed.	Date of work.	Length of pipe underdrains removed.	Length of conduit removed and relaid.	Length of pipe underdrains relaid.	Extension of gravel underdrains beyond pipe.	Approximate cost of work.
20	September 26	36 ft.	30 ft.	95 ft.	10 ft.	$240
23	June 7-8	32 "	30 "	90 "	10 "	370
24	May 31-June 1	32 "	30 "	88 "	10 "	400
25	May 25	30 "	30 "	94 "	10 "	370
Totals		130 ft.	120 ft.	367 ft.	40 ft.	$1 380

NOTE.—There should be added to this sum $880, for unused material from the 1898 work, making a total of $2 260.

Results from Renewals in 1900.—An estimate has been made by the writers to ascertain the increase in the yield of the filter due to the reconstruction of Beds Nos. 23, 24 and 25, immediately after the rebuilding was accomplished, and one month and a half later. In the first case, the data of loss of head, temperature of water, rate of pumping, and actual yield of filter, for the 5 days from May 13th to 17th, having been reduced to a uniform basis, were compared with those for the 7 days from June 11th to 17th, the surface of the filter being practically in the same condition during each period. This study indicated an increase of about 41% in the yield of the filter. A comparison of the data for the first-named period with those of a period of 8 days from July 28th to August 6th, omitting July 31st and August 2d, gave an apparent increase in yield of 17 per cent. This result was probably affected by the imprisonment of air in the sand during the early days of the last period, twelve beds having been scraped on July 27th, and the filter rapidly refilled. It was also, doubtless, affected by the sanding of several beds during the period intervening since reconstruction.

Disturbance of Efficiency by Renewals.—The question whether such work can be done to a sand filter without detriment to the health of the people receiving water therefrom is an interesting and important one. The writers believe that, if carried on with the usual care which should accompany all such work, and in suitable, but not freezing, weather, this may be answered affirmatively. It is true, however, that placing sand in a reconstructed portion, unless the greatest precautions are taken, is likely to leave it somewhat more porous than the remainder of the bed. Referring to the bacterial results obtained from the Lawrence Filter, after the work performed in 1898, it is found that, directly after the operations in October and November, the increase in the number of bacteria, including *B. Coli Communis*, in the effluent was slight, and continued for a few days only. The

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December work, however, was not carried on under as favorable temperature conditions, and even with the great care exercised, the long-continued strain of steady work upon the men may have permitted some frozen sand to be replaced in the beds. Channels were thus probably formed allowing the water to pass down without proper filtration, because the number of bacteria in the effluent was high for about three weeks after the completion of the work, and *B. Coli Communis* was found for a longer period and in greater numbers than before. In addition to this evidence, there occurred, for a few weeks after the end of the period of high bacterial numbers, a marked increase in the number of cases of typhoid fever in the city. The reconstruction work carried on in 1900 did not cause any marked increase in the number of bacteria in the effluent from the filter, and the health of the city was not apparently affected thereby.

Benefits due to Renewals.—Attention has already been called to the gain in the capacity of the filter due to the relaying of clogged underdrain pipes and gravel, and it is natural to ask: Will this gain be permanent, or will the trouble re-occur? Since the improvement, the filter has been operated in a more careful and exact manner, scrapings have been uniform and have occurred at regular periods, refilling has been accomplished less rapidly than heretofore, and the water in the pump-well has not been drawn below Elevation 31. The filter, however, has not been covered, owing to a failure to appropriate money for this purpose, and, much to the regret of the Water Board and all who have the interest of the filter at heart, the process of scraping and the maintenance in general in the winter time continue to be carried on under adverse conditions. During the extreme cold weather of the past winter, however, it has been possible to avoid operating the pumps upon Sundays, the general loss of head, with the pumps running at full speed, has not reached the maximum, and high water in the reservoir has been constantly maintained.

Results of Operations.

In undertaking to answer the several inquiries of Mr. Soper, relating to the seeming "inconsistency between the filter and its results," the writers desire it to be understood that their personal ideas alone are advanced, but it is hoped that opinions from those in touch with the early investigations and construction of the filter may be received.

At the time when the Lawrence filter was designed, daily aeration of the filtering medium was regarded as necessary, in filtering Merrimac River water, in order to maintain the effective life of the nitrifying organisms and the consequent oxidation of the polluting matters contained in the applied water. Further experience, however, has shown that the oxygen dissolved in the river water is not exhausted by the work of purification. For this reason, shortly after the filter was

placed in operation, it was learned from the bacterial results and the effect upon the health of the people, that the provisions relating to the periodical uncovering of the sand surface and drawing down of the water within the sand, "so that the sand could fill with air from top to bottom," was not only unnecessary, but inexpedient. In other words, it was discovered that the water of the Merrimac River was of such a character, as regards organic matter and the amount of oxygen dissolved, that it could be efficiently purified by continuous filtration. Messrs. Knowles and Hyde.

Mr. G. W. Fuller's discussion has already adequately shown that the design of the filter, under ordinary conditions of operations, was such as to preclude rates greater than 5 000 000 or 6 000 000 galls. per acre per day. The experience of the writers, which they understand has been confirmed by several other investigators, leads them to believe that, with not too muddy waters, good results can be secured with slow sand filters operated at rates as great or even greater than those just indicated, during the warmer seasons of the year, and possibly during all but the coldest weather. The effect of this information upon the possibility of obtaining satisfactory effluents with a filter surface the parts of which are at the same time clogged to varying degrees, thus permitting some variation in rate, is obvious.

Where suitable means are provided for carrying the applied water over the surface, without disturbance of the sand or partially formed *schmutzdecke*, back-filling with filtered water does not appear to the writers to be necessary. In the design of the Lawrence filter, advantage has been ingeniously taken of this possibility.

The essential features of sand filters for intermittent or continuous filtration are the same, the only differences being those of detail relating for the most part to processes of draining and filling, and these are not such as to be a bar to continuous operation with a water which can thus be satisfactorily purified.

An interesting feature in the design of the Lawrence filter is the provision for the greatest lateral filtration, before reaching the underdrain system, of all water passing through those portions of the sand body having least depth.

Mr. G. W. Fuller, on page 333, has called attention to a most important question, and it is hoped that students of filtration will succeed in producing some more determinative method of indicating the value of sands as filtering mediums than the present makeshift of stating the percentage of grains of certain sizes.

Expense of Maintenance.

So far as the writers are aware, the general subject of the disturbance of work by rain and the difficulty in scraping caused by light falls of snow, introduced by Mr. Bailey (page 323) and Mr. Collins (page 328), is one to which reference has not previously been made in

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the discussion of the operation of uncovered filters. This is another reason in favor of providing covers for sand filters in climates warmer than those which have hitherto been regarded as sufficiently mild for open construction. If the writers correctly interpret the general consensus of opinion, there is a growing tendency to cover filters in many cases where formerly it was considered unnecessary.

Mr. Hazen's deduction of figures on pages 313 and 314, showing the relative economy of the Lawrence filter compared with the Albany plant, on the basis of actual cost of filtration including fixed as well as operative charges, is certainly most interesting, and, to those who have not studied relative cost from this standpoint, somewhat unexpected.

The writers cannot but feel especially gratified that so many detailed figures of the comparative cost of operating certain representative American filter plants have been advanced. All these studies point to the fact that great saving can and should be effected in the cost of conveying. In all the larger filtration projects of the future, this item will doubtless receive much consideration, and it is reasonable to expect that decided advantage will be gained by the use of mechanical appliances for this work.

Chemical and Bacterial Results.

Unfortunately, so far as the authors are aware, no measurements to determine the quantity of ground-water entering the main conduit and old filter gallery have been made. If some water were derived from the river by infiltration through the embankment and through the bottom of the filter, there would be little reason to expect any increase in chlorine, or an appreciable increase in hardness or total solids. Moreover, all evidence in canal and river work at Lawrence goes to show that, in a very short time after any disturbance, the bank and bed of the stream become so completely clogged with silt as to render them practically impervious.

The area of the superficial water-shed directly tributary to the filter is so small as to preclude the possibility of any considerable portion of the daily supply coming from that source. It is quite possible, however, that the actual area of territory tributary to the filter may be much greater than the apparent water-shed. Perhaps it is the yield of water from such a district which is responsible for the very considerable difference between certain constituents found in river water and those found in the water in the pump-well. Table No. 17 shows that the hardness of the effluent has been increased beyond that of the applied water, an average of about 40%; the total solids by an average of possibly as much as 25 to 30%, and chlorine by about 9 per cent. The excess of these constituents has been less in recent years. The average amount of iron found in the effluent of the filter during 1899 is stated in the report of the Massachusetts State

Board of Health as 0.54 part per million, but the amount present in the applied water is not given. Messrs. Knowles and Hyde.

A series of samples, analyzed under the direction of the writers, was collected on July 15th, 1901, from the following sources: Merrimac River, opposite intake of filter; effluent from underdrain of Bed No. 4; effluent of entire filter; ground-water entering filter gallery near manhole. The results of the analyses of these samples are given in Table No. 57.

TABLE No. 57.

Source.	Color.	Parts Per Million.						
		Silica turbidity.	Total solids.	Loss on ignition.	Oxygen consumed.	Chlorine.	Iron.	Total hardness.
Merrimac River	.....	10	84	44	9.6	4.1	1.18	13
Underdrain, Bed No. 4	0.24	10	62	50	6.3	3.8	0.30	29
Effluent of filter	0.36	5	122	68	6.8	5.2	0.58	33
Ground-water	0.06	5	121	70	5.7	4.5	0.13	65

The samples indicate that the total solids, total hardness and chlorine are much greater in the filter effluent than in the river water. The amount of iron in solution in the ground-water as collected was less than in the effluent of the filter, and the chlorine was also lower. The total hardness of this sample was nearly double that of the effluent of the filter, but the total amount of solids contained was about the same.

It will be noted that Table No. 23, showing the free ammonia present in the Merrimac River as given by Mr. Hazen, does not accord with the figures compiled by the writers in Table No. 17. Mr. Hazen's figures represent the averages of monthly samples analyzed at the Boston laboratory of the Massachusetts State Board of Health, being the only series for the full period mentioned and used by him; while the figures used by the writers are the averages of weekly samples collected from the same place and analyzed at the laboratory of the Lawrence Experiment Station, and for the shorter period represent a greater number of samples, and thus are better to use.

In regard to the general decrease year by year in the numbers of bacteria present in the water of the Merrimac River and in the filter effluent, to which attention has been called by Mr. Soper, the writers do not feel competent to assign any definite cause. They feel confident, however, that the results for river and filtered waters for stated periods are directly comparable. Methods of laboratory procedure in bacteriological analysis of waters have undergone some change, and it is quite certain that the results given have been the best product of the times. It is known, however, that many large streams seem to have certain cycles in which less numbers of bacteria are found for a

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time; and it is also true that as a filter grows in age the effluent contains fewer bacteria, and disturbances formerly causing serious effects then seem to occasion no trouble.

Mr. Whipple's discussion is particularly interesting, and throws light on the troublesome question as to whether covered reservoirs, in which to store filtered water, are needed.

Prevalence of Typhoid Fever in Lawrence.

In Mr. Hazen's discussion of the actual benefit secured by the use of filtered water, as shown by the decreased prevalence of typhoid fever in Lawrence, the effect upon the health of the people of Lawrence caused by the introduction of a purer water supply by the City of Lowell appears not to have been directly included. The consequent very marked decrease in the number of cases of typhoid fever occurring in that place (Lowell doubtless being the chief and the most direct source of infection of Merrimac River water with the germs of the disease in question) must have been influential in reducing the sickness from typhoid fever in Lawrence, even if the river water were continued in use without purification. This fact must be regarded as offsetting in some measure the general increasing pollution of the river water during recent years. Other discussions state that the writers' estimate of usefulness of the filter is too large, and for these and other reasons the writers still believe that their original estimate of the efficiency of the filter, viewed from the standpoint of the reduction of the typhoid fever death rate, is rational. It is not unreasonable, however, also to assume that the filter has been instrumental, as Mr. Hazen suggests, in reducing the general death rate, particularly in so far as this is affected by deaths from diarrhœal diseases. But above all, in agreement with Mr. Soper's statement, the writers believe that, whatever the actual numerical estimate, the whole work and expenditure has been worth more than all it cost.

The writers heartily agree with the views expressed that "percentage of removal" is not the only criterion of efficiency of filtration, but the residual numbers of bacteria in the effluent of a filter is a more definite gauge when the influent contains large numbers. Especial significance is attached to this view, in the minds of the writers, as recent studies coming under their observation point to the fact that *B. Coli Communis*, the representative bacterium indicating sewage contamination, passes through ordinary sand filters in as great if not greater proportion than bacteria of other species with which it is present in the applied water.

The statements in the last few paragraphs of Mr. Tribus' discussion are to the point, and the writers believe that where the cost is not unnecessarily large, and under other favorable conditions, a pure supply from uncontaminated mountain sources has advantages over a purified water from a sewage-polluted stream.

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No. 907.

TRANSITION CURVES.

By W. B. Lee, M. Am. Soc. C. E.

WITH DISCUSSION.

For ordinary railroad purposes, the transition curves in common use are of about equal utility. All require tables, and, with these in hand, can be readily staked out. When, however, we are confronted with the task of designing and detailing for the shop, track-work such as turnouts and crossings, new problems arise. The multiple compound curve requires a maze of complicated computation when applied to this class of work (in which loci are absolutely necessary). The cubic parabola appeals to the writer as the simplest curve which will fulfill the conditions of a transition. But, it has been urged by some that the cubic parabola is not a true transition curve, nor can it be readily staked out with a transit.

Let us see, then, if we can more nearly approximate these prerequisites in a curve of similar character.

In developing the cubic parabola the deviation from exact values consists in substituting the differential of the abscissa for the differential of the length of the curve, in the general formula for the radius of curvature,

$$r = \frac{d\,s^3}{d\,x\,d^2\,y} = \frac{d\,x^3}{d\,x\,d^2\,y} = \frac{d\,x^2}{d^2\,y}.$$

Now, if we reverse the substitution we can say

$$\frac{d\,s^3}{d\,s\,d^2\,y} = \frac{d\,s^2}{d^2\,y}.$$

Equating with the other value of $r = \frac{R\,s_1}{s}$, and inverting, we have

$$\frac{d^2\,y}{d\,s^2} = \frac{s}{R\,s_1},$$

whence, by integrating twice, we have $y = \frac{s^3}{6\,R\,s_1}$.

This is more nearly the equation of a true transition curve, for $r = \frac{R\,s_1}{s}$ (which fulfills the condition of a transition curve, and in which R is the radius of the circular curve to be eased, and s_1 the total length of the transition curve) is now exact, and the substitution of $d\,s$ for $d\,x$ involves less error than the substitution of $d\,x^3$ for $d\,s^3$. Moreover, since it is expressed in terms of the length of the curve, it is better adapted to staking out with a transit. Before the deflection angles can be readily obtained, x must be determined. This cannot be found exactly in a finite number of terms, but, by a slight approximation, a very convenient and practical formula can be developed.

In triangles of small angle between hypothenuse and base we may express the excess of the former over the latter as follows:

Let e = excess; p = perpendicular; h = hypothenuse; and b = base.

Then $e = \frac{p^2}{2\,h}$ (nearly), for $h^2 - b^2 = p^2$, but $b = h - e$, whence $h^2 - (h^2 - 2\,h\,e + e^2) = p^2$ and $e = \frac{p^2}{2\,h - e}$; dropping $-e$, as relatively very small, we have $e = \frac{p^2}{2\,h}$ (nearly) .....(Q. E. D.)

Call the total excess of s over x, t, that is, $x = s - t$.

Then $$d\,t = \frac{d\,y^2}{2\,d\,s}.$$

Now $$y = \frac{s^3}{6\,R\,s_1}, \quad d\,y = \frac{s^2\,d\,s}{2\,R\,s_1}, \quad d\,y^2 = \frac{s^4\,d\,s^2}{4\,R^2\,s^2{}_1}.$$

Substituting, $$d\,t = \frac{s^4\,d\,s}{8\,R^2\,s_1{}^2}.$$

Integrating, $$t = \frac{s^5}{40\,R^2\,s^2{}_1}, \text{ and } x = s - \frac{s^5}{40\,R^2\,s^2{}_1}.$$

$\frac{s^5}{40\,R^2\,s_1{}^2}$ may be obtained on the slide rule to the one-hundredth

part of a foot. Any value of x being now at hand we may determine any deflection angle B by

$$\frac{y}{x} = \text{tan. } B.$$

The total angle A which is made by a tangent to the curve at any point with the initial tangent is found as follows:

$$\frac{dy}{ds} = \text{sin. } A,$$

$$y = \frac{s^3}{6\ R\ s_1},$$

$$\frac{dy}{ds} = \frac{s^2}{2\ R\ s_1} = \text{sin. } A.$$

This curve has the further advantage that, after we have fixed the constant $R\ s_1$, we can find any r (by dividing the constant by s) with a greater degree of accuracy than in the ordinary cubic parabola.

For those who would still prefer the equation $y = \frac{x^3}{6\ R\ x_1}$, the curve may be rectified by the following formula:

$$s = x + \frac{x^5}{40\ R^2\ x_1^2}.$$

It is found in the same manner as

$$x = s - \frac{s^5}{40\ R^2\ s_1^2}.$$

Notwithstanding the substitute which he has offered, the writer would still urge upon those engaged in designing layouts of track-work the use of the equation $y = \frac{x^3}{6\ R\ x_1}$, on account of the facility with which intersections may be determined by it. For example, suppose we have two transition curves leaving a common tangent and intersecting, as in Fig. 1.

$$y_1 = y_2,\ x_1 + x_2 = 100.$$

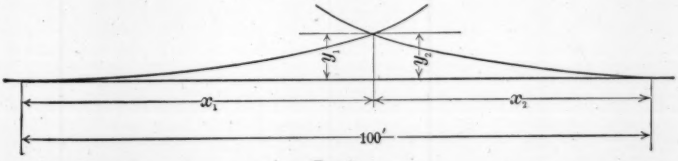

FIG. 1.

To simplify formulas let us call $R\,x_1$, the constant quantity in the equation of the curve, C_1 and C_2,

then $$y_1 = \frac{x_1^3}{6\,C_1} = y_2 = \frac{x_2^3}{6\,C_2};$$

and $$x_1 = x_2 \sqrt[3]{\frac{C_1}{C_2}};$$

substituting in $x_1 + x_2 = 100$,

$$x_2 \sqrt[3]{\frac{C_1}{C_2}} + x_2 = 100,$$

and $$x_2 = \frac{100}{\sqrt[3]{\frac{C_1}{C_2}} + 1}$$

after finding x_1, y_1 and y_2, we may find the total angle, A, for each curve, by tan. $A = \frac{x^2}{2\,C}$, and their sum is the angle of intersection.

DISCUSSION.

Mr. Snyder.

GEORGE D. SNYDER, M. Am. Soc. C. E. (by letter).—This paper is a valuable addition to the mathematics of transition curves, and the formulas given are of great practical utility, particularly in street railway work. It is unfortunate that, notwithstanding the uniform practice which exists in this country in regard to staking out railway curves, such a wide diversity should exist in the use or non-use of transition curves. In spite of the various articles which have appeared in the technical press, and the discussions by engineering societies, no general use of transition curves, or general uniformity of practice, has resulted.

All agree as to the benefits to be derived from the use of such curves, and many valuable methods of staking them out have been published, but, for some reason or other, the practical men who are called on to introduce them seem to fight shy of them as a needless refinement.

A comparison of the various methods will show that most of them differ but little, and give results that are practically uniform, within reasonable limits. The formulas are generally simple, and the application not complicated. In most methods the necessary information for use in the field has been tabulated for convenience, and the fact that these tables are more or less complicated, and limited in scope, may have deterred some from using them. As the necessary calculations are not complicated, it would seem that the use of tables is unnecessary. The author refers to the use of the slide rule for such calculations, and the writer is of the opinion that, if the use of this instrument was more general, a more general use of transition would result, as with its aid all the calculations necessary for a transition curve can be made in a few minutes, thus making tables unnecessary.

The author's formulas are well adapted for use on the slide rule. They can be applied to a method of deflections or offsets, the offset method being generally more convenient for railroad work, for the reason that the field work, field notes and plotting of the curve are but little changed from the methods in use for ordinary curves; also, for the reason that the transition curve need not be staked out in detail during the preliminary stages of the work, but merely provided for by offsetting the circular curve, toward its center from the tangents, leaving the staking of the transition curve until ready for the laying of the track.

On railroad work, using the offset method, the information required for any given degree of curve is, the length of transition curve, the offset from the tangent to the main curve when prolonged to a parallel

Mr. Snyder. tangent at the *P. C.*, and the intermediate offsets to various points on the transition curve. For this purpose the author's formula becomes:

$$Y = \frac{S^2}{24\,R}$$

In which Y = the offset to the main curve at the *P. C.*

By putting $$\frac{5\,730}{D^\circ} = R,$$

in which D° is degree of curve, we get

$$*\,Y = \frac{S^2\,D^\circ}{137\,520}$$

and $$S = \sqrt{\frac{137\,520\,Y}{D^\circ}}$$

With this formula, by a single setting on the slide rule, the offset for any length of transition curve for any given degree of curve can be obtained to the nearest one-hundredth of a foot, at a glance. Intermediate offsets, being as the cube of the distance, can likewise be obtained readily on the slide rule.

The transition curve bisects the offset Y at the *P. C.*, and is bisected by the offset line at this point. It will be found most convenient in practice to divide the half length of the transition curve into a number of equal parts, and calculate on the slide rule the offsets for a corresponding number of points. Points can then be set on half the transition curve by offsets from the tangent, and on the remaining half by offsets from the circular curve, using the same offsets in inverse order and opposite direction.

If the deflection method is preferred, the deflections, being as the square of the distance, can likewise be obtained readily on the slide rule. The preceding methods and formulas can be applied with equal facility to compound and reverse curves, and to the improvement of old lines, as well as on new construction.

The writer used similar methods on the introduction of transition curves on more than 100 miles of existing railway, and, while the methods used were not mathematically exact, no errors sufficient to be appreciable manifested themselves in the field work. In this work, stone monuments were set to mark the beginning, end and middle points of each transition curve, intermediate points being set about 25 ft. apart and marked by oak stakes.

The cost of this work may be of interest. The most expensive work, naturally, was where there was the most curvature, which was on a section about 20 miles long, 47% being curved, the average de-

**Transactions*, Am. Soc. C. E., Vol. xxxvi, p. 511.

gree of curve being 5° 00′ and the amount of curvature being 125° 05′ per mile. The cost per mile was as follows: Mr. Snyder.

Adjusting and marking center line	$17.17
Stone monuments	3.60
Labor, planting and distributing monuments	7.50
Total	$28.27

For this sum per mile a railroad can put the easing of its curves on a more scientific basis than the "crow-bar" and "rule-of-thumb" methods of the track foreman, and at the same time obtain much better results.

Some roads are expending large sums for the elimination of curvature, and many others only refrain from doing so for lack of funds. The latter may find the introduction of transition curves a profitable investment, pending the time when they can afford to eliminate their excessive curves entirely.

C. A. Sundstrom, M. Am. Soc. C. E. (by letter).—Transition curves are of great importance to the railroad engineer, and are gradually coming into more general use. These curves are also of great importance to the traveling public. For, if a railroad is provided with transition curves, there can be no shock and subsequent inconvenience to the passengers when the train is entering or leaving a curve. But, although the cubic parabola is the only curve that fully meets the conditions of a transition curve, the spiral is more commonly used. There can be no other reason for this, than that the majority of railroad engineers imagine that it is impossible to stake out a cubic parabola by means of a transit. The writer has had a great deal of experience with transition curves, and nearly twenty years ago worked up tables which will enable the engineer to stake out a transition curve, derived from the cubic parabola, just as easily as a circular curve. Mr. Sundstrom.

Before showing the use of the tables, it will be necessary to review briefly some of the theories upon which they are based.

The equation of the cubic parabola is

$$y = \frac{x^3}{6Rl},$$

in which R is the radius of the center curve, and l the length of the parabolic arc.

This being the equation of the curve obtained when a cantilever beam bends under the action of a load suspended at the free end, the cubic parabola is also called the elastic curve.

The length of the parabolic arc depends upon the elevation of the outer rail and the grade assumed in order to reach or leave the

Mr. Sundstrom. elevation. The elevation, h, of the outer rail depends upon the gauge G, the velocity v, and the radius of curvature R.

Thus $$h = \frac{G\,v^2}{g\,R} = \frac{G\,v^2}{36.16\,R}.$$

Assuming a grade of 1 ft. in n feet, the length of the parabolic arc will be

$$l = n\,h = \frac{n\,G\,v^2}{32.16\,R} = \frac{C}{R},$$

$$C \text{ denoting } \frac{n\,G\,v^2}{32.16}.$$

By differentiating the equation of the cubic parabola we obtain

$$\frac{d\,y}{d\,x} = \frac{x^2}{2\,Rl},$$

which is the natural tangent of the angle which the tangent to any point of the curve makes with the tangent of the curve.

The deflections are obtained by considering the triangle $A\ B\ C$, in Fig. 2, in which $A\ D\ B$ is the parabolic arc, the tangent of which is $A\ C$. The deflection angle, $D\ A\ C$, or d, is obtained from

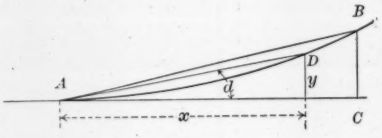

FIG. 2.

$$\text{tan. } d = \frac{y}{x} = \frac{x^2}{6\,Rl} = \frac{x^2}{6\,C}.$$

Thus the deflection is a function of the distance only, and entirely independent of the main curve.

The deflection for the point of osculation is obtained from

$$\text{tan. } d = \frac{l^2}{6\,R\,l} = \frac{l^2}{6\,C}.$$

The tables have been calculated in conformity with the following conditions: That the grade of the outer rail rises 1 ft. in 300 ft.; the velocity is 40 miles an hour, and the gauge is 4 ft. 9 ins. Consequently, the constant $C = 152314.74$. These tables can be used also for a speed of 60 miles per hour, but, in that case, the grade of the outer rail rises 1 ft. in 133.3 ft.

The tables are best illustrated by an example.

Let $A\ B$ and $A\ C$ (Fig. 3) be two tangents, intersecting at A; the angle of intersection $P\,A\,B = 120°$. Given, a 6° curve, to determine the elastic approaches.

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The first step is to ascertain whether the intersection angle is large enough for a 6° curve.

According to Table No. 1 the tangential angle for a 6° curve is 4° 46′. This is the angle $K\,F\,E$. As the angle $S\,I\,F = P\,A\,B - 2\,(K\,F\,E)$, it is evident that, when the angle $P\,A\,B = 2\,(K\,F\,E)$, the angle $S\,I\,F$ is equal to 0. In that case the points N and E coincide, and the curve consists of two elastic approaches only. When $2\,(K\,F\,E)$ is larger than the intersection angle $P\,A\,B$, the center curve must be changed to a degree of curvature having a sufficiently small tangential angle.

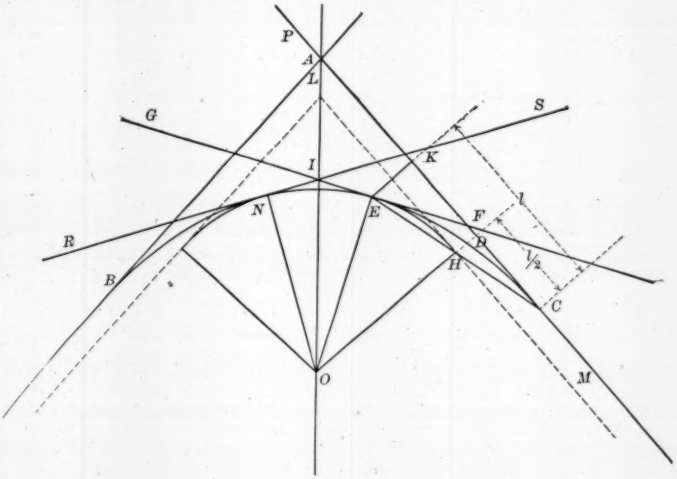

FIG. 3.

In this example, however, the intersection angle is 120°, and $2\,(K\,F\,E) = 9°\ 32'$. Hence the given intersection angle satisfies the required conditions.

We must now find the distance $A\,C$ in order to find the point of curvature.

$$A\,C = A\,D + D\,C = O\,D \text{ cotan. } O\,A\,D + D\,C$$
$$= (O\,H + H\,D) \text{ tan. } D\,O\,A + D\,C$$
$$= (R + s) \text{ tan.} \frac{I}{2} + \frac{l}{2};$$

R being the radius of the center curve; s the corresponding shift, $D\,H$; I the intersection angle, and l the length of the parabolic arc, or elastic approach.

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We have, therefore,

$$A\ C = (955.4 + 1.11)\ \text{tan.}\ \frac{120^\circ}{2} + \frac{159.4}{2} = 1736.4.$$

The values of l and s are taken from Table No. 1 for a 6° curve.

After the distances $A\ C$ and $A\ B$ have been measured, set the transit at C and sight on A, the vernier being set at zero. If stakes are to be set every 25 ft., the first deflection must be for that distance. According to Table No. 2, the deflection for 25 ft. is 3′, for 50 ft. 10′, for 75 ft. 22′, for 100 ft. 37′, for 125 ft. 59′, for 150 ft. 1° 25′. The length of the elastic approach for a 6° curve, according to Table No. 1, is 159.4 ft., which makes the last deflection 1° 36′. We have now reached the point of osculation. Set the transit over the point of osculation, E, and lay off the tangent, $F\ E\ G$; the tangential angle, $G\ F\ A$, being 4° 46′. The last operation is best accomplished in the following manner:

The angle $F\ E\ C = G\ F\ A - E\ C\ K = 4^\circ\ 46' - 1^\circ\ 36' = 3^\circ\ 10'$. The angle $C\ E\ G$, being the supplement of the angle $F\ E\ C$, is equal to 176° 50′. After the transit has been placed over the point E, set the vernier at 176° 50′, take a sight at C, and turn the instrument to zero. We have now obtained the new tangent, and are ready to lay out the main curve. Before this can be done, however, the intersection angle, $G\ I\ R$, must be found. From what precedes, this angle is equal to $P\ A\ B - 2\ (K\ F\ E) = 120^\circ - 9^\circ\ 32' = 110^\circ\ 28'$, and we stake out a 6° curve for this angle in the usual manner. After the point N is reached, move the instrument to B, and proceed in the same manner with the elastic approach $B\ N$.

When the intersection angle is small, that is to say, a trifle more than twice the tangential angle, the main curve can be shifted and provided with elastic approaches by simply furnishing the point of curve, the point of tangent, the points of osculation and the external secant. The co-ordinates for the point of osculation are obtained from Table No. 1.

Questions have arisen concerning the accuracy of the assumption that the length of the parabolic arc is equal to the abscissa of its terminus. It must be admitted, of course, that there is a slight difference. In order to ascertain the exact amount of this difference, a formula for the rectification of a cubic parabolic arc will be derived:

The length of any arc is $L = \int \sqrt{1 + \frac{d\,y^2}{d\,x^2}}\ d\,x$

The equation of the cubic parabola is

$$y = \frac{x^3}{6\ C},$$

hence

$$\frac{d\,y}{d\,x} = \frac{x^2}{2\ C},$$

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and $$\frac{d\,y^2}{d\,x^2} = \frac{x^4}{4\,C^2}.$$

Thus $$L = \int \left(1 + \frac{x^4}{4\,C^2}\right)^{\frac{1}{2}} d\,x$$

$$= \int \left(1 + \frac{x^4}{8\,C^2} - \frac{x^8}{32\,C^2} + \text{etc.}\right) d\,x.$$

As C is very large in comparison with x, the third term in the series can be discarded, and

$$L = x + \frac{x^5}{40\,C^2} + C_1,$$

for $$x = 0, \quad C_1 = 0;$$

hence $$L = x + \frac{x^5}{40\,C^2}.$$

Examining the length of a parabolic arc for an abscissa of 100 ft. it is found that

$$L = 100 + \frac{(100)^5}{40 \times (152\,314)^2},$$

or, $L = 100 + 0.011 = 100$ ft. $0\frac{1}{8}$ in.

For an abscissa of 150 ft. the arc is equal to 150 ft. 1 in.

For an abscissa of 186 ft., the largest one in the tables, the arc will be 186 ft. $2\frac{7}{8}$ ins.

For an abscissa of 200 ft., the arc is equal to 200 ft. $4\frac{1}{8}$ ins.

These figures show that, for practical purposes, the arc may be assumed to be equal to the abscissa of its terminus. Where shop work is required, the longer arcs ought to be rectified.

The foregoing refers only to the tables which have been calculated for steam railroad purposes. In the curves for street railways, as will be seen from the tables, the arcs have been rectified.

The writer has frequently had occasion to run in transition curves on street railroads, and then used the figures in Table No. 3. The convenience of this table is self-evident, and its use presents no difficulties in the shop, as the same templet can be used for all approaches, the only variable quantity being the length.

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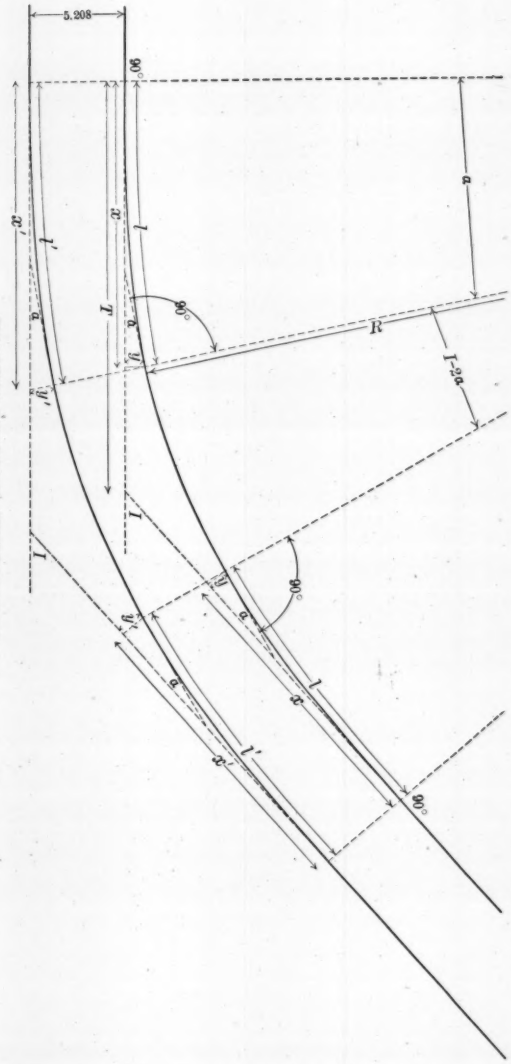
TABLE No. 1.

Degree of curvature.	Length of parabolic arc.	Shift.	Ordinate in point of osculation.	Tangential angle in point of osculation.
3°	79.7	0.14	0.55	1° 12′
5′	82.0	0.15	0.60	1° 16′
10′	84.2	0.16	0.65	1° 20′
15′	86.4	0.18	0.71	1° 24′
20′	88.6	0.19	0.76	1° 29′
25′	90.8	0.20	0.82	1° 33′
30′	93.0	0.22	0.88	1° 38′
35′	95.2	0.24	0.95	1° 42′
40′	97.5	0.25	1.02	1° 47′
45′	99.7	0.27	1.08	1° 52′
50′	101.9	0.29	1.16	1° 57′
55′	104.1	0.31	1.24	2° 2′
4°	106.3	0.33	1.32	2° 7′
5′	108.5	0.35	1.40	2° 13′
10′	110.7	0.37	1.49	2° 18′
15′	113.0	0.39	1.58	2° 24′
20′	115.2	0.42	1.67	2° 30′
25′	117.4	0.44	1.77	2° 35′
30′	119.6	0.47	1.87	2° 41′
35′	121.8	0.49	1.98	2° 47′
40′	124.0	0.52	2.09	2° 53′
45′	126.2	0.55	2.20	3°
50′	128.5	0.58	2.32	3° 6′
55′	130.7	0.61	2.44	3° 13
5°	132.9	0.64	2.57	3° 19′
5′	135.1	0.67	2.70	3° 26′
10′	137.3	0.71	2.83	3° 32′
15′	139.5	0.74	2.97	3° 39′
20′	141.7	0.78	3.12	3° 46′
25′	143.9	0.82	3.26	3° 53′
30′	146.2	0.85	3.42	4° 1′
35′	148.4	0.89	3.57	4° 8′
40′	150.6	0.93	3.74	4° 15′
45′	152.8	0.98	3.90	4° 23′
50′	155.0	1.02	4.08	4° 31′
55′	157.2	1.06	4.25	4° 38′
6°	159.4	1.11	4.44	4° 46′
5′	161.6	1.16	4.62	4° 54′
10′	163.9	1.20	4.81	5° 2′
15′	166.1	1.25	5.01	5° 10′
20′	168.9	1.30	5.22	5° 19′
25′	170.5	1.36	5.42	5° 27′
30′	172.7	1.41	5.64	5° 36′
35′	174.9	1.46	5.86	5° 44′
40′	177.1	1.52	6.08	5° 53′
45′	179.3	1.58	6.31	6° 2′
50′	181.5	1.64	6.55	6° 11′
55′	183.8	1.70	6.79	6° 20′
7°	186.0	1.76	7.04	6° 29′



TRANSITION CURVES FOR STREET RAILWAYS

Fig. 4.

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TABLE No. 2.

Deflection.	Distance.	Deflection.	Distance.	Deflection.	Distance.
1'	16.3	52'	117.6	43'	165.5
2'	23.1	53'	118.7	44'	166.3
3'	28.2	54'	119.8	45'	167.1
4'	32.6	55'	120.9	46'	167.9
5'	36.5	56'	122.0	47'	168.7
6'	39.9	57'	123.1	48'	169.5
7'	43.1	58'	124.2	49'	170.3
8'	46.1	59'	125.2	50'	171.0
9'	48.9	1°	126.3	51'	171.8
10'	51.6	1'	127.4	52'	172.6
11'	54.1	2'	128.4	53'	173.4
12'	56.5	3'	129.4	54'	174.1
13'	58.8	4	130.4	55'	174.9
14'	61.0	5'	131.5	56'	175.6
15'	63.1	6'	132.5	57'	176.4
16'	65.2	7'	133.5	58'	177.2
17'	67.2	8'	134.5	59'	177.9
18'	69.2	9'	135.4	2°	178.6
19'	71.1	10'	136.4	1'	179.4
20'	72.9	11'	137.4	2'	180.1
21'	74.7	12'	138.3	3'	180.9
22'	76.5	13'	139.3	4'	181.6
23'	78.2	14'	140.3	5'	182.3
24'	79.9	15'	141.2	6'	183.1
25'	81.5	16'	142.2	7'	183.8
26'	83.1	17'	143.1	8'	184.5
27'	84.7	18'	144.0	9'	185.2
28'	86.3	19'	144.9	10'	185.9
29'	87.8	20'	145.8	11'	186.6
30'	89.3	21'	146.7	12'	187.4
31'	90.8	22'	147.7	13'	188.1
32'	92.2	23'	148.6	14'	188.8
33'	93.7	24'	149.5	15'	189.5
34'	95.1	25'	150.3	16'	190.2
35'	96.5	26'	151.2	17'	190.9
36'	97.8	27'	152.1	18'	191.6
37'	99.2	28'	153.0	19'	192.3
38'	100.5	29'	153.8	20'	193.0
39'	101.8	30'	154.7	21'	193.6
40'	103.1	31'	155.6	22'	194.3
41'	104.4	32'	156.4	23'	195.0
42'	105.7	33'	157.3	24'	195.7
43'	106.9	34'	158.1	25'	196.4
44'	108.1	35'	158.9	26'	197.1
45'	109.4	36'	159.8	27'	197.7
46'	110.6	37'	160.6	28'	198.4
47'	111.8	38'	161.4	29'	199.1
48'	113.0	39'	162.3	30'	199.8
49'	114.1	40'	163.1	31'	200.4
50'	115.3	41'	163.9		
51'	116.4	42'	164.7		

TABLE No. 3.

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x	y	l	R	α	x'	y'	l'
4.9	0·051	4.900	128.160	1° 05′ 43″	5.000	0.020	5.000
5.0	0.033	5.000	125.597	1° 08′ 25″	5.104	0.034	5.104
9.615	0.236	9.620	65.279	4° 12′ 35″	10.000	0.250	10.006
10.0	0.265	10.006	62.761	4° 33′ 08″	10.413	0.281	10.415
10.1	0.273	10.106	62.140	4° 38′ 36″	10.522	0.290	10.524
10.2	0.282	10.206	61.531	4° 44′ 07″	10.630	0.300	10.632
10.3	0.290	10.307	60.928	4° 49′ 42″	10.738	0.308	10.741
10.4	0.299	10.407	60.343	4° 55′ 19″	10.847	0.317	10.850
10.5	0.307	10.507	59.768	5° 01′ 00″	10.955	0.327	10.959
10.6	0.316	10.608	59.199	5° 06′ 44″	11.064	0.337	11.068
10.7	0.325	10.708	58.647	5° 12′ 31″	11.173	0.346	11.178
10.8	0.334	10.809	58.098	5° 18′ 20″	11.282	0.356	11.287
10.9	0.344	10.909	57.566	5° 24′ 14″	11.390	0.367	11.396
11.0	0.353	11.009	57.043	5° 30′ 10″	11.499	0.377	11.505
11.1	0.363	11.110	56.524	5° 36′ 10″	11.608	0.388	11.615
11.2	0.373	11.210	56.020	5° 42′ 13″	11.718	0.399	11.725
11.3	0.383	11.311	55.520	5° 48′ 18″	11.827	0.410	11.835
11.4	0.393	11.412	55.029	5° 54′ 27″	11.936	0.421	11.944
11.5	0.404	11.512	54.551	6° 00′ 39″	12.045	0.433	12.054
11.6	0.414	11.613	54.076	6° 06′ 54″	12.155	0.444	12.164
11.7	0.425	11.713	53.615	6° 13′ 13″	12.264	0.456	12.274
11.8	0.436	11.814	53.156	6° 19′ 34″	12.374	0.468	12.385
11.9	0.447	11.914	52.710	6° 25′ 58″	12.483	0.480	12.494
12.0	0.459	12.015	52.267	6° 32′ 26″	12.593	0.493	12.605
12.1	0.470	12.116	51.831	6° 38′ 57″	12.703	0.505	12.716
12.2	0.482	12.216	51.407	6° 45′ 30″	12.813	0.518	12.827
12.3	0.494	12.317	50.985	0° 52′ 07″	12.923	0.531	12.937
12.4	0.506	12.418	50.571	6° 58′ 47″	13.033	0.545	13.048
12.5	0.518	12.519	50.163	7° 05′ 29″	13.143	0.558	13.159
12.6	0.531	12.619	49.765	7° 12′ 15″	13.253	0.572	13.270
12.7	0.544	12.720	49.370	7° 19′ 04″	13.364	0.586	13.382
12.8	0.557	12.821	48.981	7° 25′ 56″	13.474	0.601	13.492
12.9	0.570	12.922	48.598	7° 32′ 51″	13.584	0.615	13.604
13.0	0.583	13.023	48.221	7° 39′ 49″	13.694	0.629	13.715
13.1	0.597	13.124	47.850	7° 46′ 50″	13.805	0.645	13.827
13.2	0.610	13.225	47.485	7° 53′ 54″	13.915	0.659	13.938
13.3	0.624	13.326	47.125	8° 01′ 00″	14.026	0.675	14.050
13.4	0.639	13.427	46.770	8° 08′ 10″	14.137	0.691	14.162
13.5	0.653	13.528	46.421	8° 15′ 23″	14.248	0.707	14.274
13.6	0.668	13.629	46.077	8° 22′ 39″	14.359	0.723	14.386
13.7	0.682	13.730	45.738	8° 29′ 57″	14.470	0.739	14.498
13.8	0.697	13.831	45.404	8° 37′ 19″	14.581	0.755	14.611
13.9	0.713	13.932	45.075	8° 44′ 44″	14.692	0.773	14.723
14.0	0.728	14.033	44.751	8° 52′ 11″	14.803	0.790	14.835
14.1	0.744	14.134	44.431	8° 59′ 41″	14.914	0.808	14.948
14.2	0.760	14.236	44.113	9° 07′ 15″	15.025	0.826	15.060
14.3	0.776	14.337	43.802	9° 14′ 51″	15.136	0.844	15.173

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TABLE No. 3—(*Concluded*).

x	y	l	R	α	x'	y'	l'
14.4	0.792	14.439	43.492	9° 22′ 30″	15.248	0.862	15.286
14.5	0.809	14.540	43.190	9° 30′ 12″	15.359	0.880	15.398
14.6	0.826	14.641	42.892	9° 37′ 56″	15.471	0.899	15.512
14.7	0.843	14.743	42.596	9° 45′ 44″	15.583	0.918	15.626
14.8	0.860	14.844	42.306	9° 53′ 34″	15.695	0.937	15.739
14.9	0.878	14.946	42.017	10° 01′ 27″	15.807	0.957	15.853
15.0	0.896	15.047	41.735	10° 09′ 23″	15.918	0.977	15.966
15.1	0.914	15.149	41.454	10° 17′ 22″	16.030	0.997	16.080
15.2	0.932	15.250	41.179	10° 25′ 24″	16.142	1.017	16.193
15.3	0.951	15.352	40.906	10° 33′ 28″	16.254	1.038	16.307
15.4	0.969	15.454	40.636	10° 41′ 35″	16.366	1.059	16.421
15.5	0.988	15.556	40.369	10° 49′ 45″	16.478	1.080	16.535
15.6	1.008	15.657	40.109	10° 57′ 57″	16.590	1.103	16.650
15.7	1.027	15.759	39.849	11° 06′ 12″	16.702	1.124	16.764
15.8	1.047	15.861	39.593	11° 14′ 30″	16.815	1.147	16.879
15.9	1.067	15.963	39.340	11° 22′ 51″	16.927	1.169	16.993
16.0	1.087	16.065	39.090	11° 31′ 15″	17.039	1.191	17.107
16.1	1.108	16.167	38.844	11° 39′ 40″	17.152	1.214	17.222
16.2	1.128	16.269	38.600	11° 48′ 12″	17.265	1.238	17.338
16.3	1.149	16.372	38.357	11° 56′ 40″	17.378	1.262	17.454
16.4	1.171	16.474	38.120	12° 05′ 13″	17.490	1.286	17.568
16.5	1.192	16.576	37.885	12° 13′ 50″	17.603	1.310	17.684
16.6	1.214	16.678	37.653	12° 22′ 31″	17.716	1.335	17.799
16.7	1.236	16.781	37.422	12° 31′ 10″	17.829	1.360	17.915
16.8	1.258	16.883	37.196	12° 39′ 54″	17.942	1.385	18.031
16.9	1.281	16.986	36.971	12° 48′ 41″	18.055	1.411	18.147
17.0	1.304	17.088	36.750	12° 57′ 30″	18.168	1.437	18.263
17.1	1.327	17.191	36.530	13° 06′ 21″	18.281	1.463	18.379
17.2	1.350	17.294	36.312	13° 15′ 15″	18.394	1.489	18.495
17.3	1.374	17.396	36.099	13° 24′ 12″	18.507	1.516	18.611
17.4	1.398	14.499	35.887	13° 33′ 11″	18.620	1.543	18.727
17.5	1.422	17.603	35.675	13° 42′ 12″	18.733	1.570	18.843
17.6	1.447	17.706	35.467	13° 51′ 16″	18.846	1.598	18.959
17.7	1.472	17.809	35.262	14° 00′ 22″	18.960	1.626	19.077
17.8	1.497	17.912	35.060	14° 09′ 30″	19.074	1.655	19.194
17.9	1.522	18.015	34.859	14° 18′ 41″	19.187	1.683	19.311
18.0	1.548	18.118	34.661	14° 27′ 54″	19.300	1.712	19.427
18.1	1.574	18.222	34.463	14° 37′ 10″	19.414	1.741	19.545
18.2	1.600	18.325	34.269	14° 46′ 27″	19.528	1.771	16.663
18.3	1.627	18.428	34.078	14° 55′ 48″	19.642	1.802	19.781
18.4	1.653	18.532	33.887	15° 05′ 10″	19.756	1.832	19.899
18.5	1.680	18.636	33.697	15° 14′ 34″	19.869	1.863	20.016
18.6	1.708	18.739	33.512	15° 24′ 01″	19.983	1.895	20.134
18.7	1.736	18.843	33.327	15° 33′ 10″	20.097	1.927	20.253
18.8	1.764	18.947	33.144	15° 43′ 01″	20.211	1.959	20.371
18.9	1.792	19.051	32.964	15° 52′ 35″	20.325	1.991	20.490
19.0	1.820	19.155	32.784	16° 02′ 10″	20.439	2.023	20.608

Example.—Suppose it is required to place a curve of 39 ft. radius, with transition approaches, between two tangents, making an angle of 41° 10′. The nearest radius to 39, in Table No. 3, is 39.09, which in this case must be used. The table gives everything but the tangent. This is found by the following formula: $T = (R + s) \tan. \frac{1}{2} I + x - R \sin. \alpha$; s being equal to $\frac{y}{4}$. Then we have $T = \left(39.09 + \frac{1.087}{4}\right) \tan. 20° 35′ + 16.0 - 39.09 \sin. 11° 31′ 15″ = 14.782 + 16.000 - 7.807 = 22.975$. From inspection of Table No. 3 it will be found that the radius of curvature gradually decreases; thus it is, at the distance of 5 ft. from the point of curve, = 125.597; at the distance of 10 ft. = 62.761, etc. When $I = 2\alpha$, the curve will be composed of the two approaches only. If the radius selected produces an angle, α, so large that $2\alpha > I$, a larger radius must be selected, so as to get a correspondingly smaller angle α, or so that 2α becomes smaller than or equal to I.

CHARLES C. WENTWORTH, M. Am. Soc. C. E. (by letter).—When a car leaves a tangent and enters a curve there are two distinct motions imparted to it: First, one of translation about the center of the curve; second, one of rotation about its own center. Mr. Wentworth.

For the first of these motions, and they may be considered separately, a circular curve is as good as any other. In fact, any curve is, at a given point, circular, with its corresponding radius of curvature. All that is necessary to be done, then, in treating this motion, is to elevate the outer rail so that the resultant of the centrifugal force and gravity may be as near as desired to a perpendicular to the plane of the track.

To find the theoretical elevation in inches for a track of standard gauge, the formula $E = \frac{v^2 D}{1\,500}$ may be used, in which v is the speed in miles per hour and D the degree of curvature. This is merely a reduction from the usual formulas which are given in inconvenient units, and is put in form for use on the slide rule.

The second of the two motions before mentioned is that which causes the jar on entering or leaving an accurately laid out circular curve. This motion is precisely the same as if the car were mounted on a frictionless turn-table and a force applied to turn it. Applying such a force, sufficient to start the table from a state of rest to, say, an angular motion of 1° in 1 second, the same force would turn the car through a total angle of 4° in 2 seconds. This is what should be accomplished by a transition curve. The condition, then, of such a curve is that its tangential angle at any point shall be proportional to the square of the length of the curve, from the point of beginning, or *P. C.*, to the point in question.

The reason why the putting in of transition curves is left to the track foreman, who invariably does it, rather than to the engineer of location, appears to be that such curves have been unnecessarily loaded with too much mathematics for a busy man. They can be run in on the ground from either end without the use of any tables whatever.

For instance, it may be desired to use a transition curve such that the deflection for the first station of 100 ft. will be 25′ from the tangent at the *P. C.* The deflection for 2 stations, or 200 ft., will be four times this, or 1° 40′; for 2½ stations, or 250 ft. $(2\frac{1}{2})^2 \times 25'$, or 2° 36′. For any distance, the square of that distance, in feet, multiplied by 0.0025, will be the deflection, in minutes, from the original tangent.

After running the transition curve as far as desired, move the transit to the end of it, take a backsight on the *P. C.* and turn an additional angle equal to twice the deflection angle used in setting the stake at the end of the transition curve. This will put the instrument on tangent at the end of the transition curve; the tangential angle at that

Mr. Wentworth.

or any point being three times the deflection used in setting such point.

The deflection for the first 100 ft. being assumed as 25′, the degree of curvature at Station 1 (100 ft. from the *P. C.*) will be 6 times 25′, or 2° 30′. This is the basis of the degree of curvature at any point. At 200 ft. from the *P. C.*, it is 5°, and at 300 ft., 7° 30′. At any point it is 6 times the deflection for the first 100 ft. multiplied by the length of the curve, in stations, to the point in question.

After running in the main curve, the *P. T.* of the transition curve can be set by using, as a deflection from tangent, twice the deflection which would be needed to set the point at the end of the main curve from the *P. T.* Then, after moving up to the *P. T.*, the intervening stakes can be set back of the transit.

Of course, any deflection may be assumed for the first 100 ft., instead of 25′, in order to make the maximum transition curve of any desired length. Mr. Sundstrom's tables agree closely with the figures obtained as described for a transition curve, the deflection for which for the first 100 ft. is 37.6′.

The method of planning and locating transition curves given herein is inexact only in the tangential angles when the curve is of considerable length. For instance, if 12° be the degree of the main curve and the transition curve be 200 ft. long (1° deflection for the first 100 ft.), there will be no error. If 300 ft. long (40′ deflection for the first 100 ft.) the error is 2′. If 400 ft. long (30′ deflection for the first 100 ft.) the error is 5′. Nevertheless, as the total angle is correctly carried through from tangent to tangent, and as these small differences in no way affect the track, the error, if it may be so called, is immaterial. The greatest error in each case is given above, which occurs at the end of the transition curve.

Mr. Boggs.

J. I. Boggs, Assoc. M. Am. Soc. C. E. (by letter). – Transition curves are undoubtedly of some advantage where the radius is less than 600 ft., but the writer heartily agrees with the "practical" man, that they are a "needless refinement" where the radius of curvature exceeds that figure. The elevation of the outer rail is the determining factor in the easement, and it is a well-known fact that if this elevation is properly adjusted for speed and gauge, and graded back on the tangent the length of the easement, no shock will be felt upon entering the curve. The great trouble is the proper adjustment of the elevation for the varying conditions of speed and grade found on railroads. To attempt, however, to educate section men up to the niceties of the cubic parabola appears to the writer a hopeless and equally useless task. The practice of running the elevation back on the tangent from full elevation at the point of curve to zero at the point of easement, or as near thereto as practicable, is much more simple, answers every purpose of the spiral, and is readily understood by the average foreman.

Mr. Boggs.

As stated, the elevation of the outer rail is the governing factor, and this same elevation presents, at times, anomalies, refusing to be bound by printed rules or mathematical equations. The writer had two 2° curves to contend with, that had always given considerable trouble. One was on a 66-ft. grade, the other was on a level at the foot of two 53-ft. grades. The rule of the road was 1 in. elevation for each degree of curvature, or 2 ins. in these cases, while the formula, $l = \frac{G\,r^2}{32.2\,R}$, gave $2\frac{5}{16}$ ins.; neither would do. Finally, the curve on the 66-ft. grade was elevated to $2\frac{1}{2}$ ins. and the elevation graded out on the tangent for 100 ft., as near as practicable; the other was elevated $1\frac{3}{4}$ ins. and graded out on the tangent for 70 ft. No further trouble was experienced with either of them, and the writer has long since arrived at the conclusion that a study of the idiosyncrasies of each particular curve is of far more benefit than all the tables printed.

The writer's practice in staking out transition curves has been to use a simple offset, measured in with the tape line, and determined from the equation, $y = \frac{x^3}{6\,P}$. Henck's Field Book* contains all the information any engineer requires in staking out a transition curve, and Mr. Sundstrom's demonstration is but a reprint of these four pages. The writer thinks Mr. Snyder's cost, $28.27 per mile, would be excessive if applied to railways.

Extreme mathematical nicety in actual practice only serves to multiply useless labor, and increases the pay-roll without adding to the efficiency of the service. For instance, the writer has seen the end-areas of cross-sections calculated to the nearest one-hundredth of a square foot and the contents of the prism figured to a minimum by the old prismoidal formula; while, in the field work, it was permissible to hold the rod within two-tenths, and drive the stake within 6 ins. of the theoretical point. The writer digresses here simply to illustrate a "needless refinement." He holds that the nearest square foot and an average of the end-areas would have been sufficient, and, probably, would have resulted in a saving of $1 200 per annum on the pay-roll. The same thing applies, as a rule, to all this curvature discussion. Engineers are busy figuring, in the office, to the nearest one-hundredth of an inch on grades and alignments, wasting time and paper, and absorbing the stockholders' dividends in needless discussions; while on the track itself the inch which has been so laboriously figured out cannot be found, let alone the one-hundredth part of it. Curvature is pretty much of a bugaboo, anyway. With good track and good rolling stock, the traveling around curves is easy; bad track and poor equipment gives rough traveling, even on tangents.

* Edition of 1881, pp. 125 to 128, inclusive.

Mr. Alden. CHARLES A. ALDEN, Assoc. M. Am. Soc. C. E. (by letter). – Transition curves for street-railway track are now almost universally used in the United States. The cubic parabola, or its variations, would answer as well for street railways as for steam railroads, as far as theoretical considerations of alignment are concerned. But the radii of curves for the former are generally small and the transition short, requiring the rails to be curved to a template, and by machine. It is also necessary to make provision for the use of tongue-switches, as it is impracticable to make a wholly satisfactory switch having a radius greater than about 100 ft., although a certain type of switch can be made of about 200 ft. radius, having, however, certain mechanical disadvantages.

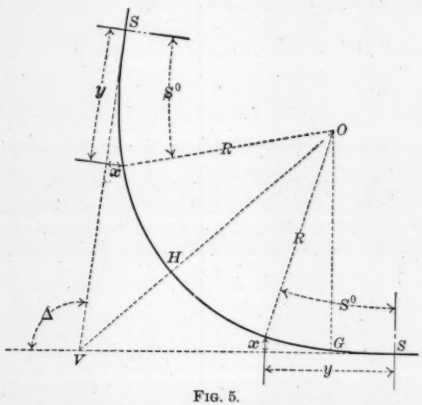

FIG. 5.

Given: A circular curve with symmetrical spirals; to find the tangent and external distances.

$O\,G = R + x -$ ver. sin. $S^\circ\,R$;
$G\,S = y -$ sin. $S^\circ\,R$;
Tan. Dist. $= O\,G$ tan. $\frac{1}{2}\,\Delta + G\,S$;
Ex. Dist. $= O\,G$ ex-sec. $\frac{1}{2}\,\Delta + x -$ ver. sin. $S^\circ\,R$.

On the somewhat rare occasions when the spirals are involved in the frog work, the calculations of intersections of cubic parabolas with each other or with a circular curve, would be quite complex, as compared with circular curves.

The calculations, in the latter case, simply resolve themselves into the solution of oblique triangles, readily adapted to logarithmic solution.

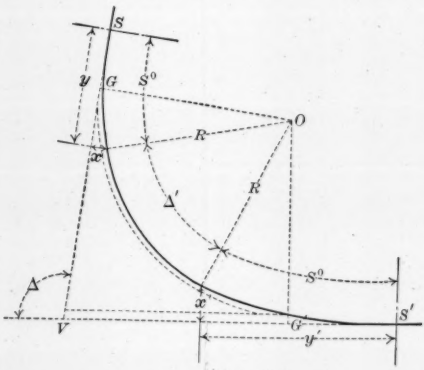

Fig. 6.

General solution for unsymmetrical curves.

$O\,G = R + x - R$ ver. sin. S°;
$G\,S = y - R$ sin. S°;
$O\,G' = R + x' - R$ ver. sin. $S^{\circ\prime}$;
$G'\,S' = y' - R$ sin. $S^{\circ\prime}$;

$$V\,S = \text{tan. } \tfrac{1}{2}\,\Delta\; O\,G + G\,S + \frac{O\,G' - O\,G}{\text{sin. } \Delta}.$$

$$V\,S' = \text{tan. } \tfrac{1}{2}\,\Delta\; O\,G + G'\,S' \pm \frac{O\,G' - O\,G}{\text{tan. } \Delta}.$$

It is necessary to have a series of transitions in order to suit the different radii met with in practice, and, at the same time, it is desirable to limit the number to those

absolutely necessary in order to avoid the expense and delay incident to calculating, laying out and preparing templates for a new transition curve, for each curve built. Mr. Alden.

Figs. 5 and 6, and Table No. 4, which explain themselves, are those used by the company with which the writer is connected. During the past year about 400 curves were built to these standards. The easements for 200-ft. radius switches are intended for, and generally used for, a Y or equilateral switch.

The other two principal manufacturers of street-railway material have standard transitions of similar form.

The writer estimates that from 70 to 80% of street-railway curves are now built according to this general form of transition.

TABLE No. 4.

Rad.	Angle.	x.	y.	$S°$	Ver. Sine.	Sine.
			Spiral No. 2.			
300	0° 30′	0.011	2.618	0° 30′	0.00004	0.00873
150	1° 00′	0.057	5.235	1° 30′	0.00034	0.02618
100	1° 30′	0.160	7.851	3° 00′	0.00137	0.05234
75	2° 00′	0.342	10.463	5° 00′	0.00381	0.08716
60	2° 30′	0.627	13.065	7° 30′	0.00856	0.13053
50	3° 00′	1.036	15.651	10° 30′	0.01675	0.18224
42½	3° 30′	1.587	18.187	14° 00′	0.02970	0.24192
37½	4° 00′	2.309	20.703	18° 00′	0.04894	0.30902
			Switch Easement, S. 2-75.			
75	7° 50′	0.700	10.222	7° 50′	This easement gives an O. G. equal to, and a G. S. 3.346 less than, Spiral No. 2.	
45½	2° 50′	1.036	12.305	10° 30′		
42½	3° 30′	1.587	14.841	14° 00′		
37½	4° 00′	2.309	17.357	18° 00		
			Spiral No. 2½.			
444	0° 20′	0.007	2.583	0° 20′	0.00002	0.00582
222	0° 40′	0.038	5.166	1° 00′	0.00015	0.01745
148	1° 00′	0.105	7.748	2° 00′	0.00061	0.03490
111	1° 20′	0.226	10.328	3° 20′	0.00169	0.05814
89	1° 40′	0.414	12.910	5° 00′	0.00381	0.08716
74	2° 00′	0.684	15.478	7° 00′	0.00745	0.12187
63½	2° 20′	1.051	18.038	9° 20′	0.01324	0.16218
55½	2° 40′	1.529	20.576	12° 00′	0.02185	0.20791
49	3° 00′	2.128	23.070	15° 00′	0.03407	0.25882
44½	3° 20′	2.870	25.550	18° 20′	0.05076	0.31454
40½	3° 40′	3.763	27.983	22° 00′	0.07282	0.37461
			Switch Easement, S. 2½-100.			
102½	6° 30′	0.658	11.584	6° 30′	This easement gives an O. G. equal to, and a G. S. 3.640 less than, Spiral No. 2½.	
56½	5° 30′	1.529	16.936	12° 00′		
49	3° 00′	2.128	19.430	15° 00′		
44½	3° 20′	2.870	21.910	18° 20′		
40½	3° 40′	3.763	24.343	22° 00′		
			Switch Easement, S. 2½-200.			
200	4° 0′	0.487	13.951	4° 0′	This easement gives an O. G. 0.250 greater, and a G S. 3.286 greater, than Spiral No. 2½.	
129	1° 0′	0.664	16.196	5° 0′		
74	2° 0′	0.934	18.764	7° 0′		
63½	2° 20′	1.301	21.324	9° 20′		
55½	2° 40′	1.779	23.862	12° 00′		
49	3° 00′	2.378	26.356	15° 00′		
44½	3° 20′	3.120	28.836	18° 20′		
40½	3° 40′	4.013	31.269	22° 00′		

NOTE.—All dimensions are for the center line of the track.

Mr. Alden.

TABLE No. 4—(*Continued*).

Rad.	Angle.	x.	y.	$S°$	Ver. Sine.	Sine.
			Spiral No. 3.			
300	1° 00′	0.046	5.236	1° 00′	0.00015	0.01745
150	2° 00′	0.229	10.468	3° 00	0.00137	0.05234
100	3° 00′	0.639	15.688	6° 00′	0.00548	0.10453
75	4° 00′	1.368	20.871	10° 00′	0.01519	0.17365
60	1° 00′	2.501	25.982	15° 00′	0.03407	0.25882
50	6° 00′	4.118	30.959	21° 00′	0.06642	0.35837
40	7° 00′	6.143	35.403	28° 00′	0.11705	0.46947
			Switch Easement, *S.* 3-100.			
102½	6° 30′	0.658	11.584	6° 30′	This easement gives an *O. G.* equal to, and a *G. S.* 4.391 less than, Spiral No. 3.	
81	8° 30′	1.368	16.480	10° 00′		
60	5° 00′	2.501	21.591	15° 00′		
50	6° 00′	4.118	26.568	21° 00′		
40	7° 00′	6.143	31.012	28° 00′		
			Switch Easement, *S.* 3-200.			
200	4° 00′	0.487	13.951	4° 00′	This easement gives an *O. G.* 0.250 greater, and a *G. S.* 2.853 greater, than Spiral No. 3.	
132	2° 00′	0.889	18.541	6° 00′		
75	4° 00′	1.618	23.724	10° 00′		
60	5° 00′	2.751	28.835	15° 00′		
50	6° 00′	4.368	33.812	21° 00′		
40	7° 00′	6.393	38.256	28° 00′		
			Spiral No. 4.			
420	0° 42′	0.031	5.131	0° 42′	0.00007	0.01222
210	1° 24′	0.157	10.261	2° 06′	0.00067	0.03664
140	2° 06′	0.439	15.384	4° 12′	0.00269	0.07324
105	2° 48′	0.939	20.490	7° 00′	0.00745	0.12187
84	3° 30′	1.720	25.561	10° 30′	0.01675	0.18224
70	4° 12′	2.899	30.567	14° 42′	0.03273	0.25376
60	4° 54′	4.352	35.469	19° 36′	0.05794	0.33545
			Switch Easement, *S.* 4-200.			
200	4° 00′	0.487	13.951	4° 00′	This easement gives an *O. G.* 0.178 greater, and a *G. S.* equal to, Spiral No. 4.	
125½	3° 00′	1.117	20.490	7° 00′		
84	3° 30′	1.898	25.561	10° 30′		
70	4° 12′	3.017	30.567	14° 42′		
60	4° 54′	4.530	35.469	19° 36′		
			Spiral No. 5.			
600	0° 30′	0.023	5.236	0° 30′	0.00004	0.00873
300	1° 00′	0.114	10.471	1° 30′	0.00034	0.02618
200	1° 30′	0.320	15.703	3° 00′	0.00137	0.05234
150	2° 00′	0.685	20.926	5° 00′	0.00381	0.08716
120	2° 30′	1.255	26.130	7° 30′	0.00856	0.13053
100	3° 00′	2.073	31.302	10° 30′	0.01675	0.18224
85	3° 30′	3.175	36.374	14° 00′	0.02970	0.24192
			Switch Easement, *S.* 5-200.			
200	4° 00′	0.487	13.951	4° 00′	This easement gives an *O. G.* equal to, and a *G. S.* 4.468 less than, Spiral No. 5.	
144	1° 00′	0.685	16.458	5° 00′		
120	2° 30′	1.255	21.662	7° 30′		
100	3° 00′	2.073	26.834	10° 30′		
85	3° 30′	3.175	31.906	14° 00′		
			Spiral No. 6.			
900	0° 20′	0.015	5.236	0° 20′	0.00002	0.00582
450	0° 40′	0.076	10.472	1° 00′	0.00015	0.01745
300	1° 00′	0.213	15.706	2° 00′	0.00061	0.03490
225	1° 20′	0.457	20.936	3° 20′	0.00169	0.05814
180	1° 40′	0.837	26.158	5° 00′	0.00381	0.08716
150	2° 00′	1.385	31.365	7° 00′	0.00745	0.12187
128	2° 20′	2.125	36.524	9° 20′	0.01324	0.16218

NOTE.—All dimensions are for the center line of the track.

TABLE No. 4—(*Concluded*).

Mr. Alden.

Rad.	Angle.	*x*.	*y*.	*S* °	Ver. Sine.	Sine.
			Switch Easement, *S*. 6–200.			
200	4° 00′	0.487	13.951	4° 00′	This easement gives an *O. G.* equal to, and a *G. S.* 7.770 less than, Spiral No. 6.	
255	1° 00′	0.837	18.388	5° 00′		
150	2° 00′	1.385	23.595	7° 00′		
128	2° 20′	2.725	28.754	9° 20′		
			Spiral No. 7.			
1 260	0° 15′	0.012	5.498	0° 15′	0.00001	0.00436
630	0° 30′	0.060	10.995	0° 45′	0.00009	0.01309
420	0° 45′	0.168	16.492	1° 30′	0.00034	0.02618
315	1° 00′	0.360	21.987	2° 30′	0.00095	0.04362
252	1° 15′	0.660	27.475	3° 45′	0.00214	0.06540
210	1° 30′	1.091	32.957	5° 15′	0.00420	0.09150
180	1° 45′	1.678	38.424	7° 00′	0.00745	0.12187
157	.....	.....	.....	.....	.....	.....
			Spiral No. 8.			
1 890	0° 10′	0.008	5.498	0° 10′	0.00000	0.00291
945	0° 20′	0.040	10.996	0° 30′	0.00004	0.00873
630	0° 30′	0.112	16.493	1° 00′	0.00015	0.01745
472½	0° 40′	0.241	21.990	1° 40′	0.00042	0.02908
378	0° 50′	0.441	27.483	2° 30′	0.00095	0.04362
315	1° 00′	0.720	32.973	3° 30′	0.00187	0.06105
270	1° 10′	1.120	38.457	4° 40′	0.00332	0.08136
236	.....	.....	.....	.....	.....	.....
			Spiral No. 9.			
2 730	0° 7′	0.006	5.559	0° 7′	0.00000	0.00204
1 365	0° 14′	0.028	11.118	0° 21′	0.00002	0.00611
910	0° 21′	0.079	16.677	0° 42′	0.00007	0.01222
682½	0° 28′	0.170	22.234	1° 10′	0.00021	0.02036
546	0° 35′	0.311	27.791	1° 45′	0.00047	0.03054
455	0° 42′	0.515	33.346	2° 27′	0.00091	0.04275
390	0° 49′	0.792	38.899	3° 16′	0.00162	0.05698
341	.....	.....	.....	.....	.....	.....
			Spiral No. 10.			
3 780	0° 5′	0.004	5.498	0° 5′	0.00000	0.00145
1 890	0° 10′	0.020	10.996	0° 15′	0.00001	0.00436
1 260	0° 15′	0.056	16.493	0° 30′	0.00004	0.00873
945	0° 20′	0.120	21.991	0° 50′	0.00011	0.01454
756	0° 25′	0.220	27.488	1° 15′	0.00024	0.02181
630	0° 30′	0.364	32.983	1° 45′	0.00047	0.03054
540	0° 35′	0.560	38.478	2° 20′	0.00083	0.04071
472	.....	.....	.....	.....	.....	.....
			Spiral No. 11.			
5 250	0° 4′	0.0035	6.109	0° 4′	0.00000	0.00116
2 625	0° 8′	0.0178	12.217	0° 12′	0.00001	0.00349
1 750	0° 12′	0.0498	18.326	0° 24′	0.00002	0.00698
1 312½	0° 16′	0.1066	24.434	0° 40′	0.00007	0.01164
1 050	0° 20′	0.1955	30.542	1° 0′	0.00015	0.01745
875	0° 24′	0.3234	36.649	1° 24′	0.00030	0.02443
750	0° 28′	0.4975	42.756	1° 52′	0.00053	0.03257
656	.....	.....	.....	.....	.....	.....
			Spiral No. 12.			
7 140	0° 3′	0.0027	6.231	0° 3′	0.00000	0.00087
3 570	0° 6′	0.0136	12.462	0° 9′	0.00000	0.00262
2 380	0° 9′	0.0381	18.692	0° 18′	0.00001	0.00524
1 785	0° 12′	0.0816	24.923	0° 30′	0.00004	0.00873
1 428	0° 15′	0.1495	31.153	0° 45′	0.00009	0.01309
1 190	0° 18′	0.2474	37.384	1° 3′	0.00017	0.01832
1 020	0° 21′	0.3806	43.613	1° 24′	0.00030	0.02443
892	.....	.....	.....	.....	.....	.....

NOTE.—All dimensions are for the center line of the track.

Mr. Howe. HORACE J. HOWE, M. Am. Soc. C. E. (by letter).—It is somewhat appalling to the busy engineer to take up each new article on the transition curve, unless he has time to go into the mathematics and see that the principles are few compared to the applications. The author has given an illustration of this by taking the well-known cubic parabola, and deriving formulas which he has found useful in his practice.

The writer's experience has been mostly on steam railroads, and, looking back at the conditions under which he labored, and under which he believes the majority now labors, it seems proper to restate the well-known but scattered conclusions which govern the practical staking out of a transition curve; to the end that all theorizers may profit thereby.

The cost of staking out should be reduced to a minimum, and, consequently, the simplest possible curve should be used.

The writer's first attempt was a series of chords or arcs beginning with a 1° curve and working up to the degree desired. Many bright engineers to-day postpone the evil day, by reserving this right to themselves. The time consumed, however, is prohibitory.

About this time, Searles published his tables, and used successive chords, with radii constantly diminishing for each chord. The objection is the disjointed character of the curve, and the labor of calculation. The treatment is too general for the special work referred to.

Next came the hyperbolic or Holbook spiral, which did specialize, with tables for only two or three cases. It had several advantages besides. The deflections vary about as the square of the distance. The backsight from the point of circular curve is double the foresight.

x_0 is one-fourth of x (nearly).

y_0 is one-half of y (nearly).

Bearing these properties in mind, it is quite possible to do without tables in the field. Can the cubic parabola do better than this?

So far as riding qualities are concerned, one easement curve is as good as another. References are made to the "true" transition curve and to the "theoretical" transition curve; but the running of a pair of wheels around a superelevated track is too complicated an operation to allow of any such claim. A good job could be done by one of half a dozen methods, and the experts would never know the difference.

Transition curves, then, should be of the simplest. They should be designed on the principle that maintenance engineers have no expense accounts; and, on that other principle, that the mental performance of any man when subject to conditions of weather is inferior to that of the same man indoors; finally, that trackmen do not throw track very often (not so often, generally, as the engineer stakes it out), and that the engineer must refresh his memory, perhaps at yearly intervals, on the subject of transition curves.

Mr. Lee. W. B. Lee, M. Am. Soc. C. E. (by letter).—To the engineer who has occasion to solve track problems in transitions, the following equations for the inside and outside rails of a curved track, the center line of which is a cubic parabola, may prove both useful and interesting:

Let $y = \frac{x^3}{6\,C}$ be the equation of the center line, g the gauge of track, y_i and y_o the ordinates of points on the inside and outside rails from the initial tangents through their own points of transition curve (*P. T. C.*), respectively; x remaining the same.

Then
$$y_i = \frac{x^3}{6\,C} + \frac{g\,x^4}{16\,C^2},$$

and
$$y_o = \frac{x^3}{6\,C} - \frac{g\,x^4}{16\,C^2}.$$

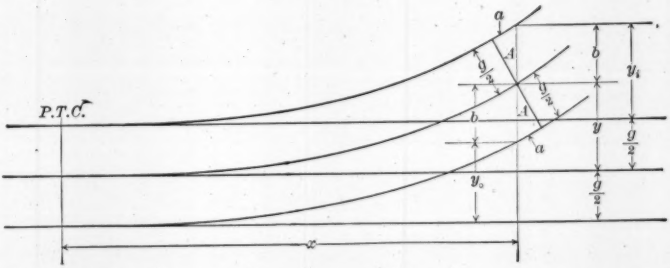

Fig. 7.

Referring to Fig. 7, they may be demonstrated as follows:

$$y_i = y + b - \frac{g}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

By the same course of reasoning as followed on page 380:

$$b - \frac{g}{2} = \frac{a^2}{2 \times \frac{g}{2}} = \frac{a^2}{g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

In the triangle shown in Fig. 7,

$$a = \frac{g}{2} \tan A. \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

from the equation of the cubic parabola,

$$\text{tan. } A = \frac{x^2}{2\,C} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

substituting (4) in (3),

$$a = \frac{g\,x^2}{4\,C} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

and
$$a^2 = \frac{g^2\,x^4}{16\,C^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Mr. Lee. substituting (6) in (2) we have

$$b - \frac{g}{2} = \frac{g\,x^4}{16\,C^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

substituting (7) in (1),

$$y_i = y + \frac{g\,x^4}{16\,C^2} = \frac{x^3}{6\,C} + \frac{g\,x^4}{16\,C^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

From Fig. 7, and similar reasoning,

$$y_o = y + \frac{g}{2} - b = y - \left(b - \frac{g}{2}\right) = \frac{x^3}{6\,C} - \frac{g\,x^4}{16\,C^2}.$$

The deflection angles, B_i and B_o, are obtained by dividing through by x, which gives

$$\text{tan. } B_i = \frac{x^2}{6\,C} + \frac{g\,x^3}{16\,C^2};$$

$$\text{tan. } B_o = \frac{x^2}{6\,C} - \frac{g\,x^3}{16\,C^2}.$$

The total angle, A, is found by differentiating;

whence,

$$\text{tan. } A_i = \frac{x^2}{2\,C} + \frac{g\,x^3}{4\,C^2}.$$

$$\text{tan. } A_o = \frac{x^2}{2\,C} - \frac{g\,x^3}{4\,C^2}.$$

It will be observed that these formulas are approximations, but, as the locations obtained from them will not vary from theoretical values more than $\frac{1}{32}$ in. in extreme cases, their practical utility is unimpaired; moreover, the gauge of the track is still exact.

In conclusion, the writer would make the following suggestions as to the selection and use of transition curves. Where there are no turnouts or crossings, and it is desired to stake out the curve with a transit, he would recommend the selection of the formula $y = \frac{s^3}{6\,C}$; for, by this, the distances around the curve may be multiples of some convenient number as 5, 10 or 25 ft.; and, as the transitman is usually more intelligent and experienced than the chainman, the former may be more readily trusted to deflect fractional angles than the latter to measure fractional distances.

For turnouts and crossings $y = \frac{x^3}{6\,C}$ is more convenient, and in switches the equation of the center line will do for both inside and outside rails. Indeed, in a simple turnout, it will also serve to give the frog lead within 3 to 6 ins. of the correct position, which is close enough for practical purposes. However, where a complicated track system exists, or it is desired to compute a set of crossing frogs, the equations for the inside and outside rails already given may be used to supplement the equation for the center line.

The writer cannot resist the temptation to digress from the subject, here, to corroborate the views expressed by Mr. Snyder, as to

the practical utility of the slide rule in engineering computations. The writer has used one constantly for the past six years and has found new uses for it daily, and new beauties in its use. At first it did not seem to be adapted to the class of work in which he was engaged, but a study of both its limits and possibilities has enabled him to use it in the majority of his computations. It relieves entirely the mental drudgery of many complicated problems, as the expert performs the several operations almost mechanically. The writer has passed through the "Mannheim" and "Duplex" to the "Universal," and he would not set a price upon his instrnment if it could not be duplicated. Mr. Lee.

This last opportunity is taken to enter a solemn protest against the use of multiple compound curves as transitions. During the summer of 1900 the writer was required to design the track work for the Boston Elevated Railroad, on which no less than thirteen different tables of multiple compound curves were in use. The memory of those computations is a hideous nightmare. One crossing alone required 1 170 sq. ins. of figures before the gauge lines could be laid down on paper.

Referring to Mr. Wentworth's discussion, too much mathematics may be an excuse for a busy "man" but not for a busy engineer. It is true that theoretical mathematicians have loaded many simple problems with a mass of unnecessary refinements; but if the engineer is well grounded in his mathematics he will be able to unearth the precious scientific truth. Popularly considered an exact science, mathematics has little more claim to such distinction than several other physical sciences, and, as all the works of man are incomplete, so mathematics shares the faults and imperfections of human nature. The writer also agrees with Mr. Boggs, that "extreme mathematical nicety in actual practice only serves to multiply useless labor," but what is permissible variation in field work would be gross error in the shop. Each engineer must study the conditions of his own work, and make his computations accordingly. It appears that the engineers engaged in this discussion have done so, as each one's idea of accuracy seems to conform to his specialty.

The writer will leave to those who have the opportunity to observe actual conditions, the question of the necessity of transition curves, but, if railroad engineers insist upon their use, let them furnish the shop men with the simplest and most practical alignment.

The complication of trackwork, needlessly, is a too-frequent practice, among railroad engineers, when a little study might simplify it. For instance, on a scale of 40 ft. to the inch it is easy to take a curved ruler and draw a crossing, introducing a set of slip switches, but it is quite another matter to compute, detail, and manufacture this work. The cost of drawings and labor in the shop is doubled, and the maintenance of the track is increased considerably by reason of the

Mr. Lee. curves. A slight change of alignment on either side of such a crossing might allow it to be straight, and thus obviate these difficulties.

The writer is glad to know, from Mr. Alden's discussion, that the use of transition curves in frog work is rare, but cannot agree with him, that "the calculation of intersections of cubic parabolas with each other or with a circular curve, would be quite complex as compared with circular curves." The writer knows whereof he speaks when he makes the statement that the computation of frog work in transitions made up of circular curves is very complex, and believes that in the paper he has partially demonstated the fact that cubic parabolas are much simpler. Should a cubic parabola intersect a circle, the equation of the latter might be taken as $y = \frac{x^2}{2R}$, except for large values of x, when it might be necessary to use the exact equation $y = R - \sqrt{R^2 - x^2}$, or $2Ry - y^2 = x^2$.

Mr. Sundstrom's formula for rectifying the cubic parabola is the same as the writer's, but has been deduced in a more orthodox manner.

A table of offsets from a tangent to a cubic parabola may be obtained with a minimum amount of labor by observing the law of the series. Such a list forms a cubic series, of which the following fundamental series, the first term of which is 1, is an example:

Series	1	8	27	64	125
First order of differences	7	19	37	61	
Second order of differences	12	18	24		
Third order of differences	6	6			
Fourth order of differences	0				

It is only necessary to compute the first term of the series. Then the first term of the first order of differences is seven times the first term of the series; the first term of the second order of differences is twelve times the first term of the series; and all the third differences are six times the first term of the series. This completes the first inclined column, from which, by addition, as many offsets may be obtained as are required. The writer* uses similar methods for marking templets of circular curves, calling the equation of a circle $y = \frac{x^2}{2R}$.

* *Engineering News*, April 29th, 1897.

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No. 908.

THE CONSUMPTION OF WATER IN MUNICIPAL SUPPLIES AND THE RESTRICTION OF WASTE.

An Informal Discussion at the Annual Convention, July 27th, 1901.

SUBJECT FOR DISCUSSION:

"The Consumption of Water in Municipal Supplies and the Restriction of Waste."

By Messrs. EMIL KUICHLING, GARDNER S. WILLIAMS, JOHN C. TRAUTWINE, Jr., RUDOLPH HERING, DESMOND FITZGERALD, JAMES OWEN, D. C. HUMPHREYS, J. P. A. MAIGNEN, J. N. CHESTER, OBERLIN SMITH, J. H. HARLOW, CLEMENS HERSCHEL, FRED. BROOKS, DEXTER BRACKETT, G. L. CHRISTIAN, FOSTER CROWELL, CHARLES W. SHERMAN and EMIL KUICHLING.

Mr. Kuichling.

E. KUICHLING, M. Am. Soc. C. E.—This topic is apparently old and hackneyed, but, in consequence of the rapid increase of our urban population, the exhaustion or growing pollution of conveniently situated sources of water supply, and the greatly augmented costs of securing additional supplies of irreproachable quality, the questions of consumption and waste of water often become paramount economic issues in our large cities, and their intelligent discussion in scientific societies from time to time is sure to be of interest not only to the members, but also to the public. New points of view are likewise de-

Mr. Kuichling. veloping, with greater experience and study, whereby convictions or opinions which were formerly regarded as well founded may now be modified or changed; and, as many of our members have doubtless accumulated valuable data relating to the topic since it was last formally debated by them, it is earnestly hoped that they will kindly present the same on this occasion.

As the sharp definition of the terms used greatly facilitates the consideration of any subject, it becomes desirable for each speaker to state as clearly as possible what he understands or includes in the expressions "consumption" and "waste." This is very necessary, inasmuch as certain items of consumption may fairly be regarded as legitimate use in one case, and unjustifiable waste in another. Ample evidence of this fact is afforded by the perusal of many old and new papers on the subject of waste prevention. Usually, waste is associated almost exclusively with the supply to families, and little is said about the quantity taken by manufacturers in various branches of industry. This may be due to following inferences drawn from European cities, where much of the water used in manufacturing is obtained from private sources, or it may arise from the circumstance that the service pipes for domestic uses greatly outnumber those for industrial purposes, and, by the multiplicity of household fixtures and faucets, afford a greater opportunity for waste. On the other hand, it is often the case that a single manufacturing establishment will consume daily more water than is required for the domestic uses of hundreds or even thousands of people, and little attention is given to the question whether a large percentage of such water is not wasted.

Inquiries of the latter kind are rarely encountered, as it is presumed that all manufacturers are served by meter and will voluntarily seek to reduce their water consumption to the lowest practicable limit. The observations of the speaker, however, lead him to the conclusion that this view is erroneous, and that in many industries a great reduction in the quantity taken from the public water supply can readily be effected at comparatively small cost to the consumer. Especially was this true in places where the water rates were relatively high, and a private supply of suitable quality for manufacturing purposes could be obtained from wells or conveniently located streams, with considerably less annual expense than was incurred for the use of the public supply. The chief objection of manufacturers to such private water plants is their initial cost and the trouble of intelligent supervision; but if the consumer can be assured of some assistance in the latter direction from the officer in charge of the public water-works, much of the difficulty will be surmounted, and an existing costly potable supply can be conserved for a considerably longer period of time than would otherwise be possible.

The terms "consumption" and "waste" accordingly need sharp

definition. By the former, the total quantity of water taken from the source is commonly understood, and hence it becomes subject to various losses of transit and storage, as well as to inaccuracies of original estimation or measurement. Where pumping engines are used as meters, large differences are often found between the actual delivery and that which is computed from the number of strokes made by the pistons or plungers. Similarly, in the case of gravity supplies, the discharge of aqueducts or pipe conduits is frequently much less some years after the completion of the works than it was at the outset; and in both cases notable variations of discharge may occur at different seasons of the same year, independently of possible leakage in transit. The loss from storage and distributing reservoirs is likewise of importance, as well as the leakage of the system of distributing pipes, stop-valves and fire-hydrants. The sum of these items is often a large proportion of the assumed or estimated total quantity taken from the source before the water reaches the consumers, and should properly be deducted before computing the consumption. In the cases where such deduction is made care should be taken to note the fact. Mr. Kuichling.

The term "waste," on the other hand, is frequently applied to all water consumed beyond a certain *per capita* quantity, which is generally determined by a more or less arbitrary comparison of the consumption in a number of communities, regardless of whether similar conditions prevail therein, and also of the quantity last in transit from the source to the consumer. Strictly speaking, however, waste should be considered only as the excess above legitimate or reasonable use, and this definition at once opens up a wide range for debate, since that which is looked upon as reasonable use in one place may be treated as waste in another. The public policy of the community, with respect to manufacturers, as well as the economical maintenance of the water pipes and fittings in buildings, are therefore factors which can fairly be taken into account, along with the cost of obtaining the required supply. In other words, a system of public water-works may sometimes be regarded as a means of promoting the rapid growth of a community, and adding largely to its wealth by making it a desirable manufacturing center, instead of being treated only as a sanitary necessity and a means for extinguishing fires. The definition of "waste" thus depends greatly on the point of view, and this in turn is largely influenced by the cost of obtaining or increasing the supply.

If a city is so situated that the expense of securing an abundance of potable water is relatively great, the legitimate uses of the supply and the restriction of waste should obviously receive the most careful consideration. In such cases, it becomes important to know what limits can fairly be placed on the domestic consumption in order to leave as much as possible for industrial uses; and, similarly, a

Mr. Kuichling. knowledge of the necessities of the various industries is required in order to prevent the abuse of an existing or projected system of works. The same is also true where the available sources of supply are of limited magnitude, as is the case with the great majority of cities. Relatively few of the large cities of Christendom are so located as to command access to practically boundless quantities of fresh water of proper degree of purity, or are so wealthy as to be able to incur the cost of conveying such water from very great distances. Lavishness of consumption is therefore an evil in most cases, and the arguments or means for abating it, as far as practicable and reasonable, deserve the widest publicity. As was tersely stated in an able paper* on this subject by the late Charles B. Brush, M. Am. Soc. C. E., presented thirteen years ago:

"The amount of water that can be used is limited; the amount that can be wasted has no limit. * * * The * * * attempt of cities and towns to increase their water supply plant to keep pace with their waste is a hopeless task. It amounts to the same thing as trying to fill a pail which has only a sieve for a bottom."

The usual prescription for the water-works authorities of a city or town wherein an excessive consumption occurs is to apply meters on every service pipe. This treatment implies that the water which is sold by measure is not wasted or used unnecessarily, and that the economical instinct of each consumer is aroused and maintained in activity by the knowledge that the meter keeps a true record of all the water drawn on his premises. It sometimes happens, however, that the same instinct leads to positive waste when the cost of preventing such unnecessary draft is greater than the value of the water; and it follows that in order to protect the works from abuse, the consumer should be compelled to place his water appliances in such condition as to prevent waste, notwithstanding his entire willingness to pay for the water drawn by him. The application of a meter, therefore, is not sufficient in such cases, as it must be supplemented by the establishment of a measure of reasonable use.

An instructive instance of this kind is afforded by the occurrence of severe cold in many of our Northern cities. On such occasions it is the practice to allow some water to run to waste during the night, in unheated rooms or premises, in order to prevent the service pipes from freezing, and during this period the aggregate normal consumption is greatly increased. A few thousand gallons thus wasted by each consumer in a week or two amounts to an insignificant sum at the usual rates, whereas the heating appliances for rendering such draft unnecessary may cost him a large amount or may even be impracticable. This condition often exists in the great majority of the buildings of our smaller cities, and when the interest on the cost of providing the water for such waste is less than that of preventing it,

* *Transactions*, Am. Soc. C. E., vol. xix, p. 89.

the extra draft becomes economically expedient and justifiable. Proper limits are, however, here again necessary. Mr. Kuichling.

The various sources of loss and waste may be classified as follows:

1.—Loss or error in computing or measuring the original quantity;

2.—Losses in conveyance and storage;

3.—Losses in distributing the supply within the corporation limits;

4.—Losses through defective pipes, fittings and appliances in buildings;

5.—Unnecessary consumption in buildings and for public purposes.

Each of these classes admits of more or less subdivision, and for each component a reasonable numerical limit can be assigned. It may also be added that interest in the topic will be greatly stimulated by accounts of the methods used in arriving at the figures adopted as standards; and it is hoped that much in this direction will be accomplished in the discussion which is now opened.

GARDNER S. WILLIAMS, M. Am. Soc. C. E.—Some diagrams are presented herewith, which, it is hoped, may be found to be of interest in connection with this subject. They represent the results of some investigations which were carried on in Detroit, Mich., during the speaker's connection with those works, and show quite clearly some of the conditions that must be taken into account in settling upon the probable daily consumption of water to be provided for. Mr. Williams

On Fig. 1 is shown the hourly consumption, as determined by plunger displacements, with an allowance of 2% for slip, on four days during 1895, which have been selected as being representative of the different conditions of service for that year. The speaker would not wish to be understood as endorsing the allowance of 2% for slip as a proper one, but as that was the allowance originally made for these engines, and as it has been adhered to in published reports, it is therefore accepted here. It will be noticed that the day begins at 2 A. M., that being about the time of minimum consumption.

February 9th, 1895, represents a typical day of extreme cold-weather consumption, and the excess of the maximum hourly consumption over the minimum is only about 18%, or the variation of consumption is about 11% above the mean and 6% below it. The total daily consumption is about 50% in excess of the average for the year.

June 18th, 1895, represents a typical day of extreme hot-weather consumption, which is the time of maximum hourly consumption and of its maximum variation, the maximum hourly consumption being 167% greater than the minimum, and 36% above the mean, while the minimum is 49% below it. The daily consumption is about 25% in excess of the average for the year.

August 1st, 1895, represents a day corresponding quite closely to

Mr. Williams. the mean day for the year. The mean daily consumption for 1895 was 40 269 731 galls., and that of the day shown was 40 577 100 galls. The maximum hourly consumption is seen to be about twice the minimum, and the variation from the mean is 49% above and 37% below. The consumption throughout the business hours of the day is seen to be quite uniform, a marked contrast to that of June 18th, in which there is a decided maximum in the afternoon, the consumption between 7 and 8 in the evening being greater than that of any hour in the forenoon.

November 10th, 1895, represents the day of minimum consumption, which was Sunday. The regular drop of the afternoon consumption is a characteristic of Sundays and holidays. Here the minimum is about 74% of the maximum, the variation from the mean being 20% above and 30% below. The total consumption on this day is about 66% of the mean for the year.

Fig. 2 shows quite clearly the effects of the daily variations of temperature and of precipitation upon the consumption of water. As at this time there was no restriction placed upon lawn sprinkling, the use of water for the purpose being without charge, it is probable that the results presented in this diagram are likely to be in excess of those in almost any other community. Meteorological conditions occurred at this time which were exceptionally favorable to a determination of the quantity of water used for sprinkling and cooling purposes. The week preceding Monday, June 3d, had been hot and dry, and each day had shown a marked increase of consumption over the corresponding day of the week before. On Monday the consumption reached 52 578 764 galls., which, when combined with the head pumped against, represented 55 313 916 157 foot-pounds of work at the engines. As the preceding day was Sunday, a day of low afternoon and evening consumption, the quantity of water pumped early Monday morning was less than would have been the case otherwise, as is shown by the record of Tuesday, in which the cross-hatched areas represent the hourly excesses of work over the corresponding hours of the day before, and it will be seen that, up to 8 A. M., Tuesday promised to be a day of greater consumption than Monday, but about this time a storm began brewing and rain fell from 9.45 A. M. to 8 P. M. in a gentle shower, the total precipitation being 0.17 in. This, and the accompanying fall of temperature, which is shown in the lower part of the diagram, caused the consumption of water to fall off rapidly, and by Wednesday it had fallen to almost exactly that of the mean day for the year. On this day's plotting the cross-hatched areas represent excess of work over corresponding hours on Monday. It seems fair to assume that the differences in consumption and work between Monday and Wednesday represent the amounts of each due to lawn sprinkling and the use of water for cooling purposes, when correc-

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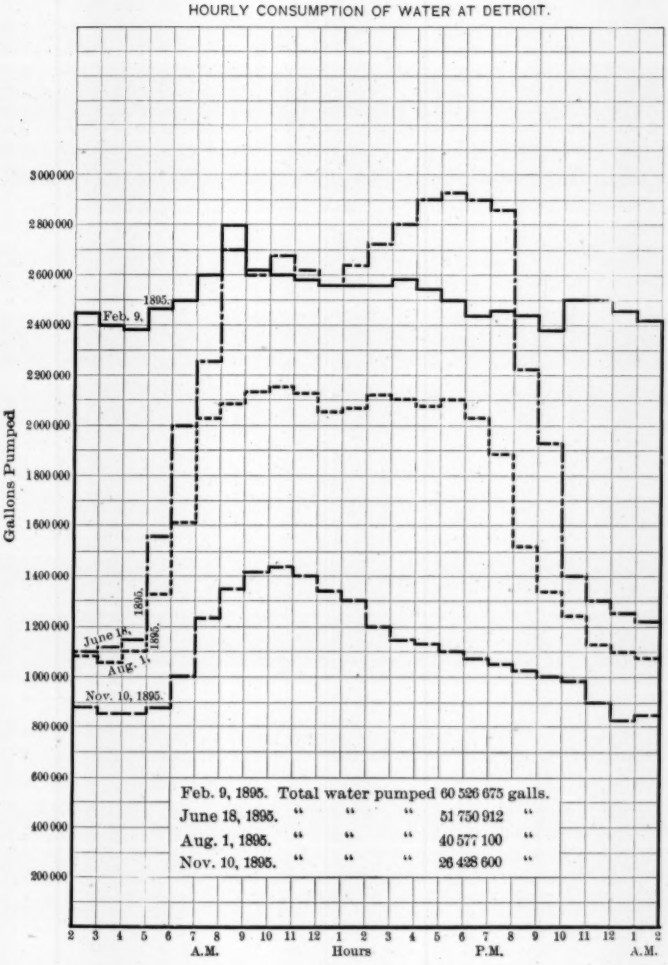

FIG. 1.

Mr. Williams.

tion is made for the fire on Monday afternoon, represented by the double hatching between 5 and 8 P. M., when four steamers were called into service; and for the variations between Sunday and week-night consumption. This gives, for the work of the engines due to sprinkling and other hot-weather uses, 15 475 131 167 foot-pounds, or more than one-third of the average daily work for the year, and the consumption of water due to these causes was more than 12 000 000 galls., or nearly 25% of the average daily for the year. The shaded area on Monday represents this excess of work, and shows clearly the distribution of such usage throughout the twenty-four hours.

These charts show conclusively that in determining the quantity of water to be provided, temperature and precipitation are elements of prime importance. To summarize the results of the studies of these effects, which have extended over several years at Detroit, during all of which time hourly records of discharge have been kept, as well as of head pumped against and pressure upon the system at some twenty different locations, it may be said:

(*a*) For a range of temperature between the freezing point and about 50° Fahr. neither changes of temperature nor amount of rainfall has any appreciable effect upon the consumption of water, and, of course, precipitation has very little effect when the temperature is below freezing.

(*b*) That for temperatures above 50° Fahr., increase of temperature increases consumption, and a decrease of temperature or an increase of precipitation decreases consumption.

(*c*) That for temperatures below the freezing point a decrease of temperature increases consumption.

(*d*) That consumption lags behind temperature changes. For instance, after high temperature the consumption will remain high until the temperature has reached a few degrees below where it was when consumption began to increase, and after low temperature the consumption will remain high until a considerable time after the mercury has passed above the freezing point.

As these matters have been more extensively discussed in the speaker's reports,* and also in an article, "Notes on Water Consumption," in *The Technic*† for 1897, where other interesting data regarding consumption in Detroit are given, the several other interesting features of these diagrams will not be considered here.

Fig. 3 represents, by the areas enclosed between the base and the upper stepped line, the mean daily consumption per consumer of water in the City of Detroit for each year from 1871 to 1899, inclusive, the total quantity of water pumped being determined by plunger dis-

* Reports, Board of Water Commissioners, Detroit, Mich., for 1894, 1895, 1896 and 1897.

† Published by The Engineering Society of the University of Michigan, Ann Arbor, Mich.

Mr. Williams.

EFFECT OF TEMPERATURE AND PRECIPITATION UPON WORK OF ENGINES OF DETROIT WATER-WORKS. JUNE, 1895.

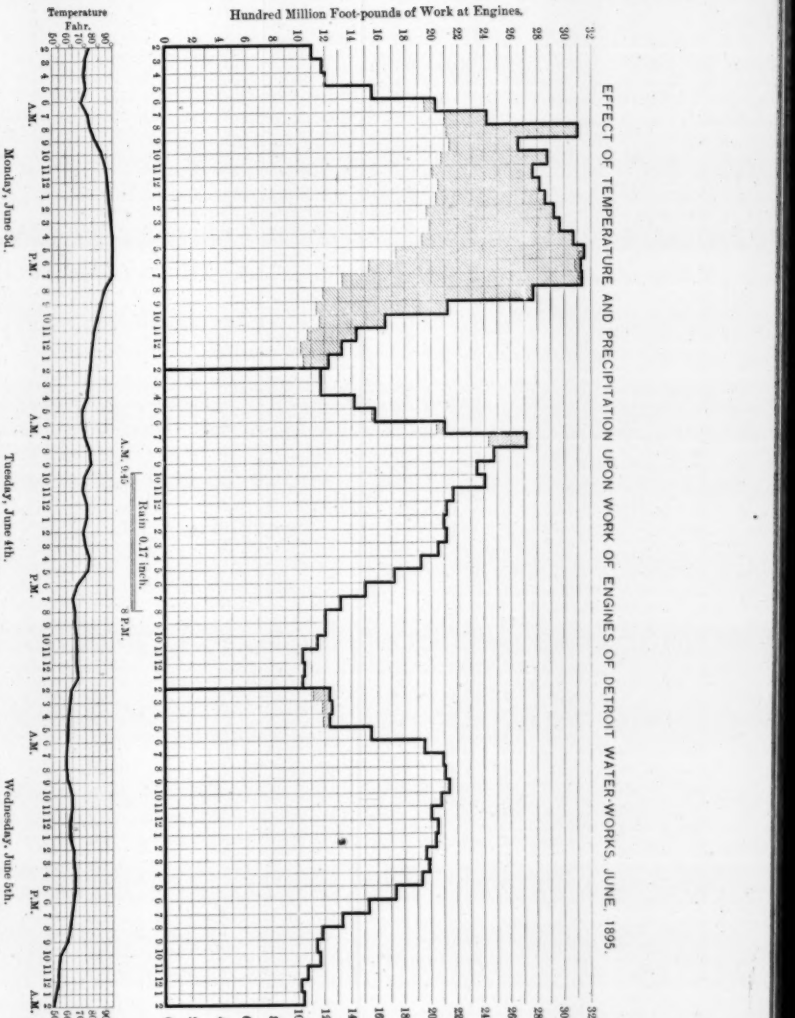

FIG. 2.

Mr. Williams.

placement as already described, and the number of consumers being obtained by multiplying the number of families using water in the city by the factor 5.14, that being the mean value of the number of persons per family, deduced from the United States censuses for 1860, 1870, 1880 and 1890. It may be remarked that owing to the modern tendency to smaller families, which is clearly shown by the later censuses, and to the concentration of population at times of financial depression and its dispersion under opposite conditions, the use of a constant factor is not strictly accurate, but the general form of the consumption area would not be greatly changed were corrections made to account for these conditions. It may be further remarked that more than 99% of the entire population of Detroit is supplied with water from the municipal works, there being no other source available for domestic use.

The shaded areas along the upper bounding line represent the average daily quantity of water per consumer sold by meter. The upper broken line represents the mean temperature for June, July and August, or the hot months, and the lower similar line represents the mean temperature for the cold months, January, February, March and December; while the lower shaded area represents the mean total monthly precipitation for the months of May, June, July, August and September, wherein rainfall may affect consumption.

On this chart the effects of temperature and precipitation can be clearly traced, where, for example, it is seen that the very low winter temperature of 1875 was responsible for high consumption, while in 1881 this appears to have been due to high summer temperature and low precipitation. In 1882 high winter and low summer temperature combined to give a low consumption, while in 1893 and 1895, and again in 1899, the opposite conditions conspired to the opposite result.

This chart is further of interest in connection with the effect of metering upon the consumption of water in Detroit. Notwithstanding that oft-quoted statements and figures from the official reports of these works are widely in error, and have been allowed to go unchallenged for several years, the speaker would not care to raise those points at this time or in this place, were it not that such statements, made by an official of those works, are on record in the *Transactions* of this Society.*

The statement is frequently made in the official publications that the reduction of consumption from the maximum in 1888 to the minimum of 1892 was due to the saving of water caused by metering. From this chart it will be seen that the quantity of water paid for by meter in 1889 was only about twice that similarly paid for in 1888,

* *Transactions*, Am. Soc. C. E., Vol. xxvi, pp. 49-52.

Mr. Williams.

ANNUAL MEAN DAILY WATER CONSUMPTION, TEMPERATURE AND PRECIPITATION DATA FOR DETROIT, MICHIGAN. 1871 TO 1899, INCLUSIVE.

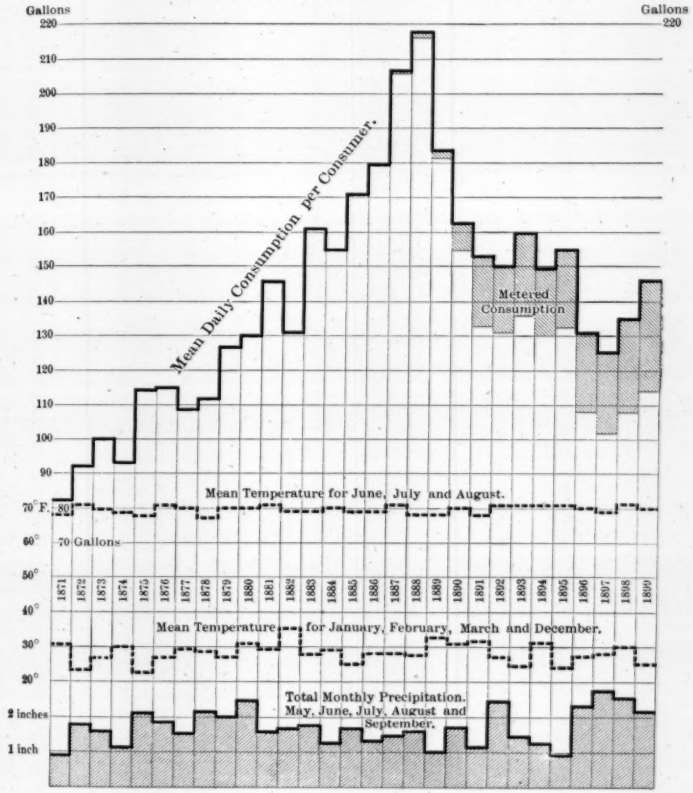

FIG. 3.

Mr. Williams. and yet the saving is nearly twice as great as during the following year, when four times as much water was sold by meter, and beyond 1890 it appears that the increase in metered water has very little bearing upon the variation of consumption. These facts are sufficient to arouse suspicions as to the remarkable efficacy claimed for the meters; and when it was found by a computation of the discharging capacity of the connections metered in 1889, that if they had all been running wide open twenty-four hours in the day in 1888 and until metered in 1889, and had been closed completely during the portion of 1889 after the meters were set, the amount they would have reduced the consumption would not begin to account for the saving claimed, it became evident that some other cause of reduced consumption was to be looked for, and upon investigation it was found that the bulk of the saving between 1888 and 1892 was due to the restriction of waste at the engines themselves, and had no connection with the meters further than that it was instituted at the same time and for the same purpose that the meters were introduced.

Previous to 1888 it was customary to open the waste-gate at the pumping engines and pump back into the river rather than lay off an engine at night when the consumption fell below a certain point. The engineer in charge of the station was very severe with his subordinates when an engine stopped, and beam engines, fly-wheel engines, are likely to stop if they run below a certain rate of speed; so when the engines were slowed down to nearly their minimum capacity the attendants opened the waste-gates, discharging back into the pump-well, to keep them pumping during the night, thus avoiding danger of trouble from having to get the chief up early in the morning to start the engines again.

It is interesting to note the decided jump in consumption from 1886 to 1887. By tracing this increase back it was found that the jump began at the time the reservoir was cut off from the system. Up to October, 1886, the Detroit system was a combination of reservoir and direct pressure, the water being pumped directly into the mains, but the reservoir being so connected that a surplus accumulated there, so that then it was not necessary to waste much water, because generally there was a chance to pump 8 or 9 ft. depth of water into the reservoir at the times when the consumption was low. But the reservoir was disconnected from the system in 1886 because it was felt that it did not afford opportunity for sufficiently high pressures, and an increased waste at the engines commenced immediately, the jump being noticeable the very first day after the reservoir was abandoned, and the waste continued to increase until, in 1888, it had become so great that the Board of Water Commissioners was appalled at the quantity of water which was being pumped, a large portion of which was never getting to the city at all. The Board feared that it would

have to reconstruct the entire system at that rate inside of five years; and the only alternative appeared to be to introduce meters, which it began to do at once. Mr. Williams.

There were some notable cases of reduction of consumption when those first meters were put in, but nothing that would account for anywhere near the drop which actually took place. Simultaneously with the introduction of the meters, the waste at the engines was curtailed, and as little water was wasted there as could be with the devices that existed for controlling the machinery, for there were times when it was necessary to waste some water. In 1893, when the speaker's connection with the works began, he had a record kept of the opening of the waste-gates, the time during which they were open, the number of turns, etc., and began estimating the quantity of water wasted in that way; and although at that time it was supposed that there was no waste, or at least that no more water was wasted than was absolutely necessary, it was found that the waste gates sometimes discharged anywhere from 500 000 to 2 000 000 galls. a day. When this waste was so great under those conditions, it can readily be imagined that considerably more had been wasted when there was no attempt to restrict the quantity thus lost. Comparisons between Sunday and week-day consumptions in 1888 showed that from 7 000 000 to 10 000 000 galls. had been wasted daily on some occasions. In 1894 the engines were remodeled, and by 1895 they were so arranged that it was unnecessary to waste. Since early in 1895 there has been no loss of water pumped at the engines in the Detroit works.

In establishing the necessary quantity of water to be allowed, it is the speaker's opinion that the range of temperature must be taken into consideration, and also the latitude of the locality in question; whether, for instance, it is in New York or in Georgia. The freedom with which the use of water for cooling purposes, for sprinkling purposes, etc., is permitted, must also be considered, as well as the character of the population. It seems impossible to lay down any hard-and-fast rule on this subject, because so many varying conditions are encountered.

There are manufacturing communities in this country where the consumption of water is reported to be only about 30 galls. *per capita.* In the City of Detroit the manufacturing and business consumption alone, all of which was metered, has amounted to more than 20 galls. per consumer, in fact to about 25 galls.; and when we reflect that in that city a large proportion of the factories draw their supplies directly from the river, and that few authorities have had the temerity to recommend 5 galls. as a proper domestic allowance *per capita,* one is led to question whether the 30 galls. *per capita* claimed in some places is not 30 galls. per inhabitant, and very possibly 60 galls. per consumer.

Mr. Williams. In June, 1896, the record of consumption of 819 private families in Detroit supplied by meter was 45 galls. per person daily, and the average daily consumption for June was less than 1% from the average for the year. These 819 families were taken at random, but probably represented mainly the middle class, and did not include more than half a dozen first-class residences. Although the speaker at one time* estimated 23 galls. per person as a proper domestic allowance for Detroit, in the light of later developments he would place the limit at nearly double that quantity.

Mr. Trautwine. JOHN C. TRAUTWINE, Jr., Assoc. Am. Soc. C. E.—Fortunately, it is no longer necessary to argue with engineers in favor of water-waste restriction, and there is practical unanimity as to the means by which such restriction may and should be accomplished; but, unfortunately, the water-works of too many American cities are managed, not by engineers, but by bodies of laymen, destitute of technical knowledge, and apt to follow the uneducated judgment of the people at large, when they are not following something less commendable. It is, therefore, in most cases, necessary for the engineer to obtain the consent of the people to be benefited in the manner he proposes, and it would seem that the only question left to discuss is, how best to obtain that consent.

For those of us who think, it is difficult to grasp the inefficacy of argument, based upon facts, in educating the public. For four years, as Chief of the Bureau of Water in Philadelphia, the speaker was instant, in season (and possibly now and then out of season) in holding up before the people the enormity and absurdity of the city's waste of water, and the deplorable consequences of that waste. He showed them that probably three-fourths of all the water pumped was being wasted, and that by a very small minority of the people, who were thus victimizing the great and stupid majority. He showed them how this waste deprived them of an ample supply and threatened the imposition of enormous additional financial burdens for no useful purpose. The people said, "We are opposed to meters." Even the grave and reverend *Public Ledger* echoed (and it still keeps re-echoing) the antediluvian cry, "Let water be as free as air," and those who condescended to use their gray matter at all in connection with the subject, asked, "How can we waste what we do not get?"

The speaker, therefore, presents his experience as a rather melancholy instance of the inefficacy of what may be called the direct or argumentative method of public education, and he would be glad to learn of more effective methods.

The speaker is glad to learn, from Professor Humphreys, that an object lesson, presented before a college of apostles, may be of use after all, and that the primer which the speaker produced before the

* *Michigan Engineer's Annual*, 1895.

American Water-Works Association, at Richmond, last year, has found its way into the hands of an abler educator, and is bearing fruit. Mr. Trautwine.

Some caution is proper in applying the data of Mr. Hering's table to the question of the advisability of installing meters in a given case. For instance, the Philadelphia works, from 1895 to 1899, were actually driven to their utmost, every pump working night and day, in the hopeless task of filling a sieve, while the Bureau of Water was besieged with complaints, personal, telephonic and by correspondence, of shocking cases of short supply. A majority of the representatives of the people, in Select and Common Councils assembled, being hostile to the existing administration, would permit no waste restriction and would grant no dollar for extensions and improvements, though they were not backward in asking for estimates of cost of filtration and extension. In response to one of these requests, the writer submitted a report, showing that, if $1 000 000 were expended upon meters, the cost of installation for filtration and other improvements would be about $3 600 000; whereas, if the waste continued unchecked, about $12 500 000 would be required. In cases like this it might be found that the introduction of meters was most desirable, even though, upon the basis of Mr. Hering's table, it might uppear uneconomical.

From the experience of Moscow it appears that an effective means of waste restriction would be the abolition of existing sewerage systems. In Moscow, a few years ago, without a system of sewerage, it was more difficult to get the water away from a house than to bring it there, and the consumption amounted to only 6 galls. per head per day.

RUDOLPH HERING, M. Am. Soc. C. E.—Mr. Trautwine's remarks express substantially the speaker's views, with, perhaps, one exception. Mr. Hering.

While all seem to be of one opinion, and should be so, regarding the advisability of preventing, so far as possible, all needless waste of the public water supply, it is necessary, in endeavoring to secure to this end the hearty co-operation of the public, to present not only a thoroughly just and practicable method, but one which will not in disguise substitute one injustice for another. Needless waste by some users, strictly speaking, correspondingly reduces the supply and its pressure for other users, or, if the supply be correspondingly augmented, it thereby increases the general expense. Herein lies, clearly, an injustice to the careful user. On the other hand, whenever it occurs in an individual case that the expense of restricting the waste is greater than the cost of furnishing the water which is saved by the restriction, then, paying by meter measurement would be manifestly as absurd as paying more than one dollar to get one dollar's worth of gold out of a mine. In this case the "careful user," who is economical and in the majority, would be forced to pay a part of the expense of a general introduction of meters, neces-

Mr. Hering. sitated by the comparatively few extravagant or wasteful users, and once more we do an injustice to the former, and establish a penalty for thrift.

A year ago, when connected with an investigation of the water supply of New York, this question was prominently brought forward. The cost of the water delivered at the city line, at an elevation of 300 ft., more or less, varied, in the several projects investigated, from $8 to $32.25 per million gallons, including interest, depreciation and maintenance.* Investigations made in Philadelphia two years ago indicated that the cost of delivering water into the city reservoirs varied in the several projects from $15.10 to $19.70 per million gallons, including interest, depreciation, pumping and maintenance.† At the pumping stations in some of our large Lake cities, the cost of water delivered under pressure per million gallons, including interest, depreciation on pumping plant and intake tunnels, pumping and maintenance is as follows: Chicago, about $11.50; Milwaukee, including the high-service lift, $12.62, and Erie, Pa., about $23. The additional cost for distribution is estimated as follows: Chicago, about $8, and Erie, including reservoir, about $16.

With these facts before him, the speaker prepared Table No. 1 in order to indicate in figures what bearing the question of the cost of water would have upon the meter question. It should be added that, of course, the complete cost of water would include not only the cost delivered into the reservoirs, but also into the buildings, and the cost of fixtures, etc. In fact, strictly speaking, it might be made to include also the resulting sewerage works, which are but a continuation and made necessary by the existence of the water supply system. For the purpose then had in view, the cost of the delivery into the existing distribution system, excluding the cost of the latter, was deemed proper. To include the cost of the distribution system would mean to add the interest on the investment, the depreciation of the pipes and appurtenances and the cost of general maintenance; then reduce the same to 1 000 000 galls. of water consumed, and find the corresponding value representing the cost of the water in Table No. 1.

The permanent cost of introducing a meter service into an ordinary dwelling-house may vary from $2 to $3 per annum. This includes the cost of installing the meter, keeping it in repair and replacing it with a new one every ten years; and includes the cost of reading it quarterly and of keeping the accounts. With a perpetual running cost of $2 or $3 per annum for metering, we should clearly expect to save thereby, respectively, at least $2 or $3 worth of water. The table supposes that

* "The Water Supply of the City of New York," by the Merchants' Association of New York, August, 1900.

† "Report on the Extension and Improvement of the Water Supply of the City of Philadelphia," by Rudolph Hering, Joseph M. Wilson, Samuel M. Gray, Commissioners; September, 1899.

Mr. Hering.

only the less amount is saved, and indicates the number of persons per meter more than which makes a meter economical, under three conditions of waste, namely, of saving 25, 50 or 75 galls. per day per person, and assuming the cost of the water to be $10, $25, $50, $75 or $100 per million gallons. If the cost of metering a dwelling-house should be $3 instead of $2, then 50% would have to be added to the number of occupants given in the table.

TABLE No. 1.

Number of Gallons which a Meter is Expected to Save per Day.		Cost of Water to City, per Million Gallons, Delivered into the Distribution System.*				
		$10.	$25.	$50.	$75.	$100.
Per head.	Per family of six persons.	Number of dwelling-house occupants, more than which makes the use of a single meter (½-in. or ⅝-in.) economical, when the perpetual cost of a meter service is not over $2 per annum.				
(1)	(2)	(3)	(4)	(5)	(6)	(7)
25.....	150	21.9	8.7	4.4	2.9	2.2
50.....	300	10.0	4.4	2.2	1.5	1.0
75.....	450	7.3	2.9	1.5	1.0	0.7

* Columns 3 and 4 might not include the cost of the distribution system.

The table is constructed from the following formula:

Let a designate the cost of the service of meter, in dollars per annum;

Let b designate the number of gallons saved by the meter service per consumer per day;

Let c designate the cost of water, in dollars per million gallons delivered by the city;

Let n designate the number of consumers using one meter.

Then let the cost of the meter service equal the cost of the number of gallons saved, or,

$$a = 365\ b\ n\ \frac{c}{1\,000\,000}$$

or, reduced to n,

$$n = \frac{1\,000\,000\ a}{365\ b\ c}.$$

Whatever quantity of water disappears by leakage, unpreventable waste on its way from the reservoir to the buildings, public hydrants, or fountains, and the quantity under-registered by the meters must, of course, be accounted for, and should be proportionately charged against private and public service, thus raising the cost per million gallons usefully consumed. Thus, unpreventable loss will vary greatly, and chiefly with the care that has been taken in the construction and maintenance of the works. In well-managed works it need not be over 15%; but even if it were 25% of all the water delivered, it would deduct but one-quarter from the number of occupants given in the table.

The table shows that, with any reasonable allowances, and in a city where water is cheap, it is not economical, as a general rule, to place meters in houses for small families, because it may then cost less to let a certain amount of water go to waste than to provide means for check-

Mr. Hering. ing it. This argument has been often made before; practical business men see the strength of it, and engineers certainly do not wish to ignore it. Unfortunately, however, it is sometimes used to oppose the introduction of meters into any house whatsoever, a conclusion altogether without reason or justice.

To deal successfully with this matter, we should, therefore, discriminate carefully, and recommend meters only for those buildings and users where the probable saving will at least pay for the meter service. And, in summing up the whole question, it appears to the speaker that the best way at present to handle the introduction of meters is:

First, to have the city or water company own the meters in all cases.

Second, to give every user the opportunity to pay for his consumption of water by meter, if he wishes to do so, and can, thereby, save himself money beyond a fixed minimum charge.

Third, to give the authorities the right to exact payment by meter measurement from any user, not only in all cases where the water is used for manufacturing or business purposes, but also in any private house when these authorities have reason to believe that the domestic consumption is greater than the quantity covered by certain fixed rates. Where water is expensive, this would mean a meter service for perhaps every house; where water is cheap, all the smaller dwelling-houses would, as a rule, require none.

It is questionable whether the greatest waste of water in a city, as shown by reported figures, is attributable to the carelessness of private consumers. J. James R. Croes, President, Am. Soc. C. E., in his examination of the New York water supply, estimates that, of the total waste, only one-fifth is inside of the houses, and four-fifths on the streets and in old taps or unused service pipes. In Philadelphia it seems probable that a similar condition exists. It appears, therefore, that in addition to the reduction of undue waste in houses, the cities should similarly examine into the economical question of reducing the waste from underground leakage.

As suggested, a district meter system can be introduced, to localize and measure leakage; then it can be ascertained, somewhat as above, whether the cost of repairing a particular leak is or is not justified by the value of the water to be saved, or by the amount of damage being done by the leak.

Another point seems to need emphasis, in view of the conclusions which have been repeatedly drawn. We frequently hear of the evidence from Fall River, Woonsocket, etc., quoted against that from New York, Philadelphia, Washington, Buffalo, etc., regarding the waste of water. The former class consists generally either of small cities, of those "having a preponderance of the operative class, whose average wealth and opportunities for luxuries" in the use of water are

small, or of those cities which are fortunate in possessing a continuance of good and vigilant officials, keeping the waste in check. The other class consists generally of cities of large population, of wealth and luxury, in which the officials have less opportunity closely to look after and control individual consumers, and in which there is sometimes a disposition to resist anything that might, though mistakably so, appear to curtail a citizen in the liberal use of all the water he needs. Mr. Hering.

Those of the latter class unquestionably and properly require a larger *per capita* supply than those of the former.

Among individual consumers, also, we almost invariably find in the houses of the wealthier class a very much greater *per capita* use than in the houses of the poorer class. It is likewise true that, along with an increase in the standard of living, from decade to decade, the legitimate *per capita* water consumption has also increased and may increase still more.

For one who has always been emphatically opposed to the waste of water and who has favored every rational means of preventing the same, it may seem contradictory to take issue with some of those engineers who desire the same end. The present question, however, is chiefly one of means, of practicable means to secure this much-desired end, and on this question of means there may be a reasonable difference of opinion.

Referring to the writer's opinion, that meters should generally be omitted in houses for small families in those cities where water is very cheap and whenever in consequence thereof a meter service would cost more than the water saved, Mr. Crowell, in his discussion, disagrees, but does not adduce any fact or clear reason why he cannot agree to this proposition. To enlarge somewhat upon this matter, the writer does not believe, nor did he ever believe, that meters should in any case be prohibited. He is desirous of seeing the waste of water, and, therefore, of money, reduced as far as it is economical. To this end he believes that the city should have the privilege to meter every house, but should place a meter only wherever it pays the city to do so; and, further, the user should have the same privilege, and should be entitled to demand the placing of a meter whenever it pays him to do so. In both cases, public and private, this is virtually a recommendation to introduce, not to prohibit, meters; and, as the writer further believes, this recommendation in both cases is based on a fundamental principle of equity and true economy, and therefore practicable. So long as we insist on metering every house, even when it can be shown to entail a loss of money, we are likely to defeat our very object, and cause the people to postpone, if not permanently reject, the more general introduction of meters.

When there is so much difficulty in convincing certain classes of

Mr. Hering water users that metering their supply would save them money, instead of the contrary, and in convincing some city governments that a more general introduction of meters would reduce operating expenses and postpone the day for enlarging their works, an endeavor should be made to throw no obstacles in the way of the best means of accomplishing this much-desired result. We should, on the contrary, try to make the way easy and smooth, and should use a rational method which, from its fairness, will appeal directly to those who must pay the bill, and which will gradually but surely obtain the wished-for result.

Mr. Crowell refers to his article on "The Use and Waste of Water in New York City." He expresses therein his opinion that in Manhattan and the Bronx a total supply of 107 000 000, or 55 galls. per head per day of the resident population, would be a sufficient allowance, instead of the present consumption of these districts, which is over 227 000 000 or 117 galls. per head per day of resident population; the difference being a theoretically preventable waste of 120 000 000 galls. per day. He estimates "the possible saving," that is, "the extent to which the prevention (of waste) can be carried," "as a very considerable part of the theoretical amount."

John R. Freeman, M. Am. Soc. C. E., states* his conclusions on the practical chances of reducing the present *per capita* supply in the City of New York, so as to postpone the inauguration of a new supply, as follows:

"It is dangerous to public interests to be too hopeful about preventing waste when estimating the date when the new supply must be available, or in estimating its necessary magnitude, for with these hopes unfulfilled and the reservoirs empty, the disaster would then be beyond remedy."

The Engineering Committee of the Merchants' Association, referred to by Mr. Crowell, in view of all the evidence before it, unanimously shared Mr. Freeman's views, and advised taking immediate steps for an increased supply as the most proper practical solution of the question, not believing it to be safe to rely on a material reduction of waste.

In view of the enormous floating population of New York, the foregoing figures given by Mr. Crowell are somewhat misleading, because, if the actual number of water consumers, including the large number of transients constantly in New York, is considered, the resident *per capita* rate, and therefore the apparent waste, would be correspondingly reduced.

In his discussion, Mr. Crowell takes exception to the statement "so often heard," that the *per capita* water consumption increases generally as wealth and luxury increase. He does not substantiate his

* "Report on New York's Water Supply, Made to Bird S. Coler, Comptroller," by John R. Freeman, March, 1900.

exception by statistics. The writer believes that this would indeed be difficult, unless there was a metering for substantially every house and a comparison in each one with the wealth of the residents, as measured perhaps by the property tax or the rent. So far as this has been done, in the writer's experience, the result has roughly indicated such a relation between the average consumption per user and the average rental of the dwelling, provided extreme cases and trade water are excluded. This result has been noticed in Europe as well as here, but here, also, no statistics seem to be available. Mr. Hering.

If the figure representing the entire delivery of water is divided by the figure representing the entire resident population, thus getting the so-called *per capita* supply, making no deduction for that part of the population which is not supplied, nor considering the transient population which is supplied, nor taking special account of the quantity of water used for manufacturing purposes, the above relation between wealth and consumption may naturally become much distorted.

It may not be amiss to emphasize again, that in large cities meters may not be the chief remedy against waste. There is in some cities a very large loss of water outside of the buildings, as pointed out frequently. Old service pipes and mains furnish a large proportion of the waste, and the resulting leakage needs correction fully as much as the waste in dwellings.

The under-registration of meters is another question that should have a careful consideration. When tested, their slip is known. After running for some time, the slip is known to increase, and a case, once before the writer, revealed, under a light flow, a slip of about 50 per cent.

From what is said above, the wastage inferred from meter records, therefore, is not always actual, except in rare cases. It may sometimes be as delusive as that recorded for pumpage when the actual slip of the pumps is not accurately known. Or, it may be deduced from a delusive figure as to the actual water consumers. Or, the loss of water by leakage in streets and on public grounds may in part tacitly be assumed as being due to waste by private consumers.

DESMOND FITZGERALD, Past-President, Am. Soc. C. E.—The speaker believes in taking every reasonable step to stop needless and willful waste of water. Here is a shining example of the beneficent results arising from the general introduction of meters. Recently, the speaker was consulted by a city in regard to an additional supply of water, and in the course of his investigations found the following remarkable conditions: Population, 40 063; consumption *per capita*, 29.3 galls.; domestic use, metered, 10.9 galls., and unmetered, 2.5 galls.; manufacturing, 4 galls.; street sprinkling, 2.5 galls.; fountains, 1.3 galls.; unaccounted, 8.1 galls. The latter includes water used for fires, flushing sewers and Mr. FitzGerald.

Mr Fitzgerald. water mains and puddling trenches. One of the interesting facts in connection with this city is that there has been no increase *per capita* in the consumption during the past six years. The total number of meters in use is 4 300; total number of services, 5 275, and about one-third of the whole supply is used for manufacturing purposes. The water for street sprinkling and for fountains is measured.

In order to check the small waste unaccounted for, the speaker made measurements of the fall of the water in the stand-pipe between the hours of 1.00 and 5.00 A. M., and found it to check almost exactly with the foregoing amount of 8.1 galls. *per capita* unaccounted for. Probably this loss is due to leakage in the pipes or unmetered connections, as the legitimate use of water in the early morning must be very small.

It is evident, on investigation, that in this case everyone uses all the water he desires, and, as there is an excellent sewerage system, the plumbing is on a liberal scale for a city of this size. The contrast between the foregoing excellent administration of a public water supply and that prevailing where no attention is paid to the restriction of waste is refreshing.

Mr. Owen. JAMES OWEN, M. Am. Soc. C. E.—The pressure under which water is delivered is a potent factor in the checking of waste. In the average small town which is compelled to supply water for fire purposes without the aid of a steam fire-engine, it will be found that the daily consumption of water is at least 20 to 30% more than in those cases where water is delivered under the ordinary house pressure, and without the requirements of fire service. This would show that the loss, as Mr. Herschel says, is behind the meter, to a large extent. Incidentally, also, there is a great loss through the fixtures, due to a higher pressure.

Another point, which may savor of heresy, is that the engineer in charge of a water plant should control the supply, to a certain extent, if he can. The speaker does not think that the public is educated up to the idea that they should not have all the water they can get, but they should be educated up to the idea that they can use only what is given them. That has been a matter of practice, secretly, in a great many cities where the service supply has been very short and where the gates have had to be closed in order to curtail the supply. In one case, during a shortage of water, the speaker made a practice of shutting off nearly all the water. A by-pass was put in by which the ordinary 12-in. main was reduced to a 2½-in. feeder. This practice was carried on for about two months before anyone found it out. The result was that the night consumption during hot weather was reduced to a minimum, and the period of dry weather was safely passed. In regard to such cases the speaker suggests that it may be possible for the engineer in charge of a water supply to arrogate to himself, perhaps,

the control of this service according to the conditions. It is too often the case that the engineer tries to give the people all there is. Mr. Owen.

In another instance a serious epidemic, resulting in many cases of sickness, was caused by the effort of the engineer to give an alternate supply, of water which was not proper to be used, in order to keep up the service and the pressure. If he had used what he had during the day and had simply reduced the pressure at night he would have had no trouble.

D. C. HUMPHREYS, M. Am. Soc. C. E.—The speaker has made very effective use of the arguments given in Mr. Trautwine's paper, before the American Water-Works Association at Richmond, Va., in educating his fellow-citizens and the town council in the town in which he lives. In that town, some twelve years ago, the speaker put in an extension of the water-works, and since then he has been striving to obtain the introduction of meters in order to check the waste, but has found it to be a long and serious task. Mr. Humphreys.

The situation is as follows: The town has a population of 3 000 to 4 000 and the supply is obtained from springs and brought by gravity to a reservoir having a capacity of about 20 000 000 galls. The pressure from the reservoir is sufficient for fire purposes without the use of engines The town has taken a great interest in its fire company—the champion fire company of the State—and the result of this, and the excellent water supply, is that the fire insurance companies have reduced the cost of insurance to the minimum rates, which is appreciated greatly by the business men.

When the reservoir was built a waste gate was provided, and, as the supply was larger than the demand, a great deal of water was allowed to run to waste. Knowing this, the people naturally became extravagant. Everyone used as much as he wanted, and said, "What is the use of saving it?" In the fall of the year, however, the springs, generally, run low, and there is no longer a surplus; all that comes to the reservoir is needed. Last fall all the water was drawn out, and then Mr. Owen's scheme of shutting off the supply was adopted, which was not very pleasant for those having lawns or for those who had been accustomed to having the streets sprinkled.

A severe lesson is probably needed in order to emphasize the necessity of curtailing waste. If a fire occurred at a time when the water supply was low, the fire company would be unable to do anything and the insurance companies might put up the rates and say, "You have not the fire protection we assumed you had when we gave you low rates." That would probably make an effective impression.

The question of meters, as a help in eliminating water waste, is no longer in dispute by engineers. The main point is to get the public to appreciate their advantages. Those who put in meters in this town wanted to put in the cheapest they could get, as they had to purchase

Mr. Humphreys. them. The town should furnish the meter, keep it in order, and charge an annual rental. A meter should be put in wherever there is a water-closet, but in the poorer houses, having only one spigot, inspection alone should be sufficient to check waste.

Mr. Maignen. J. P. A. MAIGNEN, Assoc. Am. Soc. C. E.—There is a point which the speaker tried to raise two years ago at the Cape May Convention, which has not yet been alluded to in this discussion, and which he would like to bring forward for consideration. A double supply exists in Paris. One is plain, unfiltered river water under a comparatively low pressure; the other is spring water and filtered water which is pumped to higher reservoirs, and which is, therefore, under greater pressure.

The plain river water is sold at 12 cents per thousand gallons, and the spring and filtered water at 24 cents. If, on account of its pressure, anyone wants to use this latter water for elevators, carriage washing, industrial or other base purpose, the charge is 48 cents per thousand gallons. This is practically a fine to prevent the use of spring water for anything but dietary or other noble uses.

Streets are washed with the plain river water from hydrants with a hose that delivers enough water to wash all the dirt into the surface sewers. It is not a mere sprinkling, which leaves the mud on the streets, it is a thorough wash-out. It is evident that for such a purpose it is not necessary to have sterilized water or any great pressure.

It has been stated that in some little towns, in warm weather, the springs do not give enough water. Every now and then we hear that New York and other great cities are threatened with water famine. This would not occur if the best water were confined to nobler uses; common water being used for street washing, lawn sprinkling, fire, industrial and other indifferent uses; the better water, pumped under a higher pressure, being reserved for house use. The pipes for the purpose would probably never require duplicating or replacing by larger ones, as the population tends, not to increase on a given area, but to spread itself on larger areas; the population, particularly with increased conveying facilities, spreading itself farther and farther into the suburbs.

On the contrary, for industrial and municipal service, the needs may outgrow the capacity of the original pipes. In that case, larger or duplicate pipes may be installed to supply the increased demand. Such water can be given away, practically as the Philadelphians want it, free like air.

In some towns of France, along the coast, where pure water is scarce, sea-water is used for washing the streets, fire extinguishing, etc. The same has been done in several American towns.

Where the plain water is very roily, it might be subjected to a rough kind of filtration at a high rate, which would not be expensive.

It certainly seems to be a mistake to use sterilized water for all purposes, particularly with such enormous consumption and waste as is current in this country. Mr. Maignen.

J. N. CHESTER, M. Am. Soc. C. E.—Every water-works should have an income sufficient to pay its operating expenses, the interest on its investment and an additional amount sufficient to meet the cost of administration, whether that administration is by the officers of a private corporation or by the mayor and council of a city. The rates of any water company, whether flat or by meter, should be regulated accordingly; and, while the speaker heartily agrees with all that Mr. Hering and Mr. Herschel have said, he thinks that they are at variance in taking the revenue, one for the cost of supplying water and the other the total cost of operating the plant. Others, also, look purely upon the cost of pumping water, and not on other operating expenses and the interest on the investment. The speaker does not believe, however, in riding the horse of economy to death, which may be done by too rigid metering. A few years ago a striking example of this might have been seen in two suburbs of New York City, along the Sound. In one every tap was metered, and in the other very few were metered. In the fully metered town there were a few very wealthy people, and quite a number in moderate circumstances, but in that town there were very few well-kept lawns except those of the wealthy people; the poorer class and those in ordinary circumstances feeling that to water their lawns and keep them looking well was a burden. In the town where water was sold at the flat rate, almost everybody sprinkled their lawns. If a meter is put on, of course the waste is curtailed, and, on the other hand, in cases where the charges are materially changed by metering, the use of water for certain purposes is sometimes curtailed. At the ordinary charge of $6 per year for sprinkling a lawn, if the lawn be of any size, and the street in front be kept well watered, it will be found that at meter rates considerably more than $6 worth of water will be used in a year. On the other hand, if a water-closet is metered and is kept from leaking, it will be found that it uses scarcely more than $2 worth, though, with the exception of the lawn sprinkler, the water-closet is believed to be the source of greatest waste. A striking example of that occurred in Shreveport, La., not long ago, where a negro cabin, in which the only fixture was a water-closet, was metered. In this cabin one dollar's worth of water a day, at 30 cents per thousand gallons, was being used. That case is not exceptional. Mr. Chester.

The speaker thinks that the question of metering is largely one of locality. In some of the western cities if meters were adopted generally the lawns would be almost ruined. In places where nothing but alfalfa will grow, and that only by irrigation, and where the inhabitants are poor, it would be ruinous to put in meters.

Mr. Chester. It has been shown that to a certain extent metering will curtail the waste, and beyond that there is very little saving in consumption. That is the case in most places, and it is found that metering factories, saloons, hotels, livery stables and such places will stop the greater part of the waste, and that in most of our northern cities the consumption will not be excessive, but can be kept below 100 galls. per head, which is looked upon as an economical consumption.

Tables made up miscellaneously are misleading. Take, for instance, the case of Birmingham, Ala., where the supply to three industries is more than the consumption of that city would be at 100 galls. per head. The consumption in such a place cannot be compared with that of other places, and, while all the industrial establishments, saloons, livery stables, etc., are metered, the consumption is higher than that of any city supplied by the speaker's company.

There is a difference in the consumption in northern and southern cities. The colored citizen is not an extensive user of water, but he is an extensive waster. He seldom sprinkles his lawn, and does not care how dusty it is in front of his cabin. He is not noted for cleanliness, and never has a bath tub. Ordinarily, he only wants water in his house when the sanitary regulations of the city require him to put it in. The case of Shreveport, La., might be cited again as an example. In that city an ordinance compels every citizen within 320 ft. of a sewer and water main to put in a connection. Of the population of that city, 60% are colored. There is scarcely one of the colored population who utilizes the water or puts in a fixture other than the spigot which he is entitled to if he puts in a water-closet and a connection thereto; and yet that town shows a greater waste per head than most cities supplied by the speaker's company. That waste is determined by the minimum pumpage at night. The speaker has always said, and especially while he was the representative of a pump company, that few engineers appreciate the variation in consumption between the hours of 4 A. M. and 4 P. M., and between the ordinary spring and fall months, and the extreme cold weather; that is, where the temperature, as shown by Mr. Williams' diagrams, ranges between freezing and 50° Fahr., and where the precipitation is about uniform. On assuming charge of the Shreveport plant, the enormous waste was first brought to the speaker's attention by the fact that the pumpage at night was equal to that in the daytime, a condition which existed in no other plant in which the supply was direct, or through a stand-pipe, which is practically direct pressure. That brought out very clearly the fact that a great deal of water was being wasted, and an inspection showed that it occurred mostly among the poor or colored families, and came largely from the water-closets. The proposal to put meters upon those places was met with tremendous resistance by the political element, or the city government, which looked to those col-

ored people for votes just as much as it looked to the white element. One councilman, with three in his family, and with a house and lawn of a size which gave him a rating of $80 per year, refused to vote for an ordinance allowing meters to be put in miscellaneously to curtail waste, saying that it would work hardship on the poorer people, and, while he felt that it might reduce his water rent from $80 to less than $15, he must decline to vote for the ordinance on account of the poorer people. Mr. Chester.

OBERLIN SMITH, M. Am. Soc. C. E.—One point does not seem to have been brought out very strongly, and that is, whether it would be possible, and if so, how far possible, to regulate publicly the quality of plumbing fixtures. It is admitted that many of them are bad, but it is not realized how bad they generally are, and it is a question whether, by proper ordinances, and perhaps by having a commission in charge of the inspection and licensing of such apparatus, its quality might not be improved, and only good, durable, permanent apparatus be allowed. This is, at any rate, a direction in which this Society and the Water-Works Associations might move, to some extent. Mr. Herschel has spoken very disparagingly of paternalism, but the speaker believes in it in some cases—where it does good and where money is to be made by it. If a man is a good father and has a bad boy he gives him a spanking, and there paternalism is a very good thing. Mr. Smith.

Public regulation of the public is needed to a greater extent than obtains in this country especially. There is not enough of it in regard to many things of a public nature; and allowing anybody to put in the worst and most unserviceable plumbing fixtures, which do not get any regulation afterward, is a bad thing, and there should be some way of getting an improved practice, gradually, at least, if it cannot be done any faster. In the wash-basin at the hotel this morning, the speaker found a stream the size of a lead pencil running, and it had been running ever since yesterday morning, and the morning before. On attempting to shut it off it was found that the tap would not close. This is happening all around us; streams of water are running uselessly from all kinds of fixtures, and no one seems to have any very particular charge of it. Of course, the putting in of meters will improve the behavior of consumers in this respect. In general, there is no doubt that meters are a good thing, although there are cases where, as Mr. Hering suggests, they will not pay. The water companies, or the municipalities, should manage and keep in good order the whole plant as a plant, as has been mentioned, including the meters as a part of the plant. But we should deal properly with the consumer and sell him things at proper prices.

One of the points which should be emphasized particularly, is

Mr. Smith. the great importance of checking underground waste until the ideal system of the future comes—that of running all pipes in galleries where they will be accessible. The chief remedy that has here been suggested is the more frequent use of large meters in the mains, and the dividing of them into separate sections, so as to trace up these serious underground escapes.

Mr. Harlow. J. H. HARLOW, M. Am. Soc. C. E.—There are one or two facts which should not be forgotten. One is, that water is to be provided at the lowest rate and sold at some price which will justify the sale. There is no free water. The speaker has never seen it, and he has been in the business for thirty years.

On the question of the introduction of meters, a company with which the speaker is connected recently engaged to supply two small towns in which, in proportion to the population, or to the takers, an enormous amount of water was used. The consumption ran from 500 000 to 600 000 galls. per day, and the rates from $12 to $15 per year for a house of the ordinary kind, with ordinary fixtures and conveniences.

The company discussed the manner in which meters could be introduced without antagonizing the consumers, and made a schedule which brought the possible rate down to about $6 per year, leaving the taking of a meter optional with the consumer. Each consumer was to pay for at least 1 000 cu. ft. of water per quarter, and the company was to furnish the meter at a certain rental, or the consumer might furnish and own the meter.

The result has been that within a year the consumption has been reduced to less than 200 000 galls. per day, and the profit to the company is as great as it was before, while the expenses are much less. It is doubtful whether the consumers in these towns could be induced to go back to the old way.

Mr. Herschel. CLEMENS HERSCHEL, M. Am. Soc. C. E.—This is a subject with which the speaker has grappled in a practical sort of way for some years, not exactly, perhaps, as an engineer, but as a wholesaler of water. There are certain impressions, and a certain amount of ignorance in the public mind, which a society like this should endeavor to remove. There is not so much necessity for the education of the members themselves, for they all are pretty well posted. It was largely for this reason that, finding the American Water-Works Association was going to meet in New York this year, and having been a member of that Association for ten years, without ever having seen them, the speaker prepared a paper on this subject which was read before that Association lately in New York City, the members of that Association being very near to the public on this general subject.

The speaker's idea of consumption is that it is composed of waste and use; his idea of waste is, everything that is up stream from the

meter; his idea of use is, everything that the consumer chooses to pay for. The speaker does not believe in a paternal form of government; he believes that the average shrewd manufacturer would resent governmental interference with his business, and would consider that he did not need a wet-nurse to tell him how to get water for the least money. It is not practicable to endeavor to teach manufacturers, unless they want to be taught, and whenever they do want to be taught they have the means at hand by consulting the profession. There is so much of salient evil in this whole matter that the minutiæ of the business may well be left for future treatment. Such is the vast amount of water wasted up stream from the meter. The regular idea, the standard notion, prevailing, not only in the public mind, but in the minds of civil engineers, is that a leak is a thing that shows on the surface; and if it does not show on the surface it is not only out of sight, but also out of mind. There was a reason for this, years ago, when there were no large meters built for large pipes. That reason no longer exists. Such a meter has been in operation in a 9-ft. pipe, and there is nothing to hinder the same meter from working in a tunnel full of water—the Niagara tail-race tunnel, if you please. Such meters do not cost very much, and they have been in practical use for eight years. Anyone who has never tried a meter on a main and seen what it reveals to him, and the methods and helps it reveals to him in his business, has yet a great many pleasant experiences before him. It is just as easy to put a meter on a district of a city, or a whole city, now, as it has been for the last twenty or thirty years to put it on a single house. Those kinds of meters are the ones by the use of which it is possible to find leaks in broken mains and in service pipes, which latter, in the speaker's opinion, are the greater sinners of the two in the way of producing waste that does not show on the surface. It is the easiest thing in the world for a main to be broken completely and run a full stream of water and nobody know anything about it for days and months and years. There was a case of this kind in Cambridgeport, Mass., about twenty years ago. There the streets are very often built up of oyster shells—or used to be, when oysters were more plentiful than they are now—and one can readily see that the flow from a 6-in. pipe, under a moderate pressure, such as also was customary then, broken off in a dump made of oyster shells, would never show on the surface. Such a one was found, and it had been leaking nobody knew how long. Mr. Herschel.

It is the same with service pipes. The speaker had a little experience of that sort in his own yard. There is a barn about 150 ft. from his house, and an extraordinary consumption of water led to an investigation by the water company, which showed that the whole house supply could be shut off, but as long as the pipe leading to the barn was open, a large quantity of water—it amounted to about $36 a

Mr. Herschel. year—went in the direction of the barn all the time, as shown by a close observation of the meter. The flow through the meter was uniform, and yet no water had ever shown on the surface of the ground. The reason was that the pipe was like a sieve throughout its whole length; and the point to be brought out about this pipe—it was a galvanized iron pipe, had been down about 15 years—is, that had it not been for the meter, it never would have been possible to suspect or find the leak. This is a single case. Imagine, now, a city like New York, and managed in the same way—imagine the many service pipes there, which are probably doing the same thing, it being remembered that New York, for a very large part of its surface, is built on very coarse sand. Apply the lesson to almost any city, and it will teach the same thing.

The same example teaches another lesson. As many probably know, the last eleven years of the speaker's life have been spent in building up a wholesale dealer in water, called the East Jersey Water Company. It sells water to municipal corporations by the million gallons. It also sells water to what is called the Orange Water Company by the million gallons. The Orange Water Company did not believe in meters until they themselves bought by meter, and from the moment they bought by meter, it took no great time to show them that they would better sell by meter. It has worked the same way in Bayonne, N. J., and similarly in the City of Newark, which in spite of its great growth now consumes less water than it did four years ago. It will work the same way anywhere.

In this matter, the speaker believes in appealing to human nature, and letting the results work out for themselves; applying business principles to what is a business operation. The furnishing of water costs money to the furnisher or seller, and if he will deal with it as other commodities are dealt with, the lesson will spread to other consumers. Nothing else is necessary, certainly not at the present time, when the gross abuses must be dealt with first.

A discussion of this sort would be very incomplete without mention of the work that Mr. Bailey, of Albany, has done within a short time, the results of which were published in *Engineering News* of April 18th, 1901. It is no longer a subject for doubtful premises and of theoretical or of arithmetical consideration to say what American cities can do by metering water to the consumers. That question is settled, at least in its general outlines. Mr. Bailey procured the statistics of 137 American cities and found that the average consumption in these cities was 137 galls. per consumer per day, but once let 50% of the taps be metered, and this consumption reduces, on the average, to 62 galls. In Mr. Bailey's table the cities were given alphabetically, but in Table No. 2 twenty-four of these cities have been arranged in their order of least consumption per consumer.

TABLE No. 2.—TWENTY-FOUR AMERICAN CITIES, ARRANGED IN ORDER OF LEAST CONSUMPTION PER CONSUMER. Mr. Herschel.

No.	City.	Consumers, when known; otherwise population.	Taps.	Percentage of taps metered.	Consumption per Day.	
					Per tap.	Per consumer.
1	Woonsocket, R. I.	32 000	2 347	91.01	398	29
2	Brockton, Mass.	35 000	5 275	81.51	221	32
3	Fall River, Mass.	104 500	6 943	94.25	548	36
4	Sioux City, Iowa.	33 111	3 000	50.83	476	43
5	Quincy, Ill.	36 252	2 700	48.15	611	46
6	Racine, Wis.	29 102	4 916	24.41	279	47
7	Lexington, Ky.	26 369	1 500	99.47	833	47
8	Des Moines, Iowa.	62 139	7 499	57.19	400	48
9	Malden, Mass.	33 664	6 426	45.67	265	50
10	Utica, N. Y.	56 383	7 064	96.80	425	53
11	Providence, R. I.	187 300	21 566	82.60	470	54
12	Topeka, Kan.	33 608	2 900	18.97	655	56
13	Lawrence, Mass.	57 200	5 926	74.01	541	56
14	Covington, Ky.	42 938	5 300	78.30	472	58
15	Manchester, N. H.	56 987	5 500	67.27	636	61
16	Dayton, Ohio.	85 333	12 000	46.67	442	62
17	Kansas City, Mo.	163 752	13 000	40.00	?	62
18	Peoria, Ill.	56 100	6 058	6.90	578	62
19	Taunton, Mass.	26 100	4 502	40.69	360	62
20	Newton, Mass.	32 800	7 087	84.68	294	64
21	Lynn, Mass.	73 000	12 577	19.88	370	64
22	St. Paul, Minn.	125 000	17 093	28.15	488	67
23	Worcester, Mass.	113 000	13 292	94.31	596	70
24	Salem, Mass.	35 700	6 400	2.81	390(?)	70

The average amounts to about 400 galls. per tap. Per consumer, it can be stated that it should be about 60. Anything that is not above 60 is fair, although in Providence, R. I., which compares very well with any city in the United States in general make-up, a city of 200 000 people, a manufacturing city with water-works more than 25 years in use, the consumption, it is believed, has never been 60 galls. from the beginning, certainly never for a long time or much over 60 galls , and is now 54 galls.

In the same way, in Table No. 3, twenty-four American cities are arranged in the order of greatest percentage of taps metered. Such a table, of course, is entirely different from Table No. 2, but is instructive. Bayonne, N. J., which stands at the head of this list, is a place of comparatively few inhabitants, and when works like those of the Standard Oil Company locate in such a place, they overslaugh the consumption by the inhabitants, and in this particular case, they produce a consumption fully 50% greater than if there were only the ordinary amount of manufacturing.

Mr. Herschel. TABLE No. 3.—TWENTY-FOUR AMERICAN CITIES, ARRANGED IN ORDER OF GREATEST PERCENTAGE OF TAPS METERED.

No.	City.	Consumers, when known, otherwise, population.	Taps.	Percentage of taps metered.	Consumption per Day.	
					Per tap.	Per consumer.
1	Bayonne, N. J.	32 722	3 000	100.00	1 024	94
2	Lexington, Ky.	26 369	1 500	99.47	833	47
3	Yonkers, N. Y.	45 000	4 968	97.67	730	81
4	Utica, N. Y.	56 383	7 064	96.80	425	53
5	Worcester, Mass.	113 000	13 292	94.31	596	70
6	Fall River, Mass.	104 500	6 943	94.25	548	36
7	Atlanta, Ga.	65 000	9 275	91.64	587	84
8	Woonsocket, R. I.	32 000	2 347	91.01	398	29
9	Newton, Mass.	32 800	7 087	84.68	294	64
10	Providence, R. I.	187 300	21 566	82.60	470	54
11	Brockton, Mass.	35 000	5 275	81.51	221	32
12	Covington, Ky.	42 938	5 300	78.30	472	58
13	Atlantic City, N. J.	28 500	4 249	77.62	587(?)	80
14	Pawtucket, R. I.	80 000	8 293	76.80	799	82
15	Lawrence, Mass.	57 200	5 926	74.01	541	56
16	Milwaukee, Wis.	300 000	41 485	67 59	578	80
17	Manchester, N. H.	56 987	5 500	67.27	636	61
18	Des Moines, Iowa.	62 139	7 499	57.19	400	48
19	Lincoln, Neb.	25 000	3 550	56.34	564	80
20	Fitchburg, Mass.	25 000	4 437	54.70	676	120
21	Lowell, Mass.	94 000	10 634	52.53	752	85
22	Sioux City, Iowa.	33 111	3 000	50.83	476	43
23	Toledo, Ohio	65 384	11 888	50.26	652	119
24	Quincy, Ill.	36 252	2 700	48.15	611	46

In such a matter of statistics as indicated in these tables, it should be emphasized that it is numbers that tells. The statistics of 137 cities will yield a true lesson, even though a great many of those statistics may be found faulty. The number of them and the law of averages makes their resultant lesson true.

The speaker has had no such experience as Mr. Trautwine has recited. On the contrary, he knows that he has been the cause of several cities or companies introducing meters, simply by selling water to them by meter. That is all that is necessary. The principle involved is not one of science at all. There is nothing in it but human nature. The fact is, no individual or city knows or thinks what it is doing, or can know or think, until the water is measured, both coming in and going out, and to be successful the lesson must be made to appeal to the pocket. There has been a vast amount of nonsense spoken on the subject of meters as instruments of punishment, or as something cruel and unusual, and that the consumer should put the meter in or pay for it on a separate bill. These are notions in which we have grown up. Now, what is a meter? What is the proper object of a meter? Here are certain fixed charges that must be met. Let the season be hot or cold, rain or

shine, those fixed charges are going on, and have to be met at the end of the year. Who is to pay them? They should be equitably distributed among the beneficiaries. The cost of the water, at the service pipe of the consumer, let it be what it will—and ordinarily this will be from $50 to $100 per million gallons—should be equitably distributed among the consumers. A meter is nothing more than an instrument for accomplishing this object, and, in water-works operation, the question as to whether or not a meter should be put in should never have been raised. The meter should have been considered as part of the plant, just as much as a gas meter is a part of a gas plant or an electric meter part of an electric plant; and probably the question never would have come up, and all this discussion never would have been had, if water meters, as perfect as they are to-day, had originally been on hand. The speaker thinks that is the only reason, and that the time for any of those considerations has gone by. Large meters and small meters are now so well built—the ordinary house meter is a marvel of good construction, durability and cheapness—that the speaker does not think the question with regard to them would ever have arisen had they been available thirty years ago in their present shape, any more than it arose with regard to gas meters or electric meters; and the time for considering them as anything else than part of the plant has gone by. Mr. Herschel.

But the greatest benefit that the method herewith described, for enlisting the interest of every member of a community in the requisite struggle against waste of water, may confer on that community is to reduce enormously the cost of the water-works themselves, or of water-works extensions. The average city in the United States may thus postpone the necessity of an impending water-works extension some 10 years. In the case of a city known to the speaker, where the leakage was measured, it appeared that by gradually introducing consumers' meters and stopping street waste until the conditions now prevailing at Fall River, Mass., had been attained, that city could get along with its existing water supply for 51 years, notwithstanding its natural growth during that half century. On the other hand, if matters were allowed to proceed as hitherto, it would need water-works extensions within a year.

Speaking on the basis of an educated judgment in such matters, the speaker estimates that Buffalo could, merely by repression of waste of water, do without water-works extensions for 30 or 40 years; the Metropolitan District of Massachusetts, for 10 or 15 years; Cincinnati, for some 20 years; and in that proportion elsewhere in the United States. There is no reason whatever, except a neglect properly to attend to that branch of water-works administration here and now under consideration, that would necessarily make any one of the named cities consume more water *per capita* than, for example, Providence, R. I.

Mr. Brooks. FRED. BROOKS, M. Am. Soc. C. E. (by letter).—An example of small works in which meters have been successfully applied is at Reading, Mass., where the population is a little less than 5 000, the water has to be purified as well as pumped, and the capacity of the source is not greatly above present needs. About 800 meters were added, at a cost of nearly $10 000, with the result of bringing the year's expenses at the pumping station below those of a previous year, selected for fair comparison, by about $700. It was estimated, from the rate of increase in the demand for water before the meters were put in, that, without them, an additional force would have been required at the pumping station for 150 days in the year at $10 per day, amounting to $1 500, or about 15% of the cost of the additional meters. The particulars are given by Superintendent Lewis M. Bancroft, in the Tenth Annual Report of the Water Commissioners of the town, for the year ending December 31st, 1899.

Mr. Brackett. DEXTER BRACKETT, M. Am. Soc. C. E.—The point referred to by Mr. Kuichling, that the recorded consumption is often much greater than the actual quantity delivered to the mains, should be emphasized. The recorded consumption is in many cases calculated from the displacement of the plungers of the pumping engines, without making sufficient allowances for the slip or loss due to defective valves and worn plungers. When pumping machinery is new and in good order, the slip is not generally more than 2% of the theoretical discharge; but an engine which was operated for several months with a slip of nearly 50% came under the personal observation of the speaker, and it is probable that many cases exist to-day where the slip exceeds 10 per cent.

Quite a large percentage of the increase in the consumption of water may be attributed to the very great increase in the number of water fixtures *per capita*, as well as to the design of the fixtures now in general use.

Table No. 4, showing the number of fixtures in use in Boston for ten-year periods from 1870 to 1900, with the percentages of increases and the increase in population and consumption during the same periods, illustrates very forcibly the great increase in the conveniences offered for using and wasting water.

While the population has doubled, the number of water fixtures in use has increased 4½ times, and the water-closets, bath-tubs and wash-tubs have increased from 5½ to 11½ times the number thirty years ago. To-day, water fixtures are very much more generally used in the cheaper class of houses than in former years, and the fixtures used are very often of inferior quality, and for that reason soon become sources of waste.

There is often a tendency to consider all the increase *per capita* in consumption as preventable waste, but it must be remembered that

new uses for water are continually arising, and although the use may be a luxury rather than a necessity, it is one which the engineer must consider as legitimate. Mr. Brackett.

TABLE No. 4.

	1870.	Percentage of increase.	1880.	Percentage of increase.	1890.	Percentage of increase.	1900.	Percentage of increase, 30 years.
Taps and sinks.	70 009	50.4	105 309	38.6	145 943	38.	201 360	187.6
Bowls.....	25 652	86.9	47 946	39.2	66 725	58.6	105 889	312.6
Bath-tubs.....	9 811	85.7	18 220	83.5	33 431	110.9	70 521	618.8
Water-closets.....	28 227	102.9	57 278	74.1	99 731	57.7	157 320	457.3
Urinals.....	2 499	65.1	4 126	22.4	5 052	13.1	5 714	128.7
Wash-tubs.....	9 615	107.3	19 930	124.9	44 816	150.9	112 435	1 069.4
	145 813	73.4	252 809	56.5	395 698	65.1	653 189	348.0
Total population....	278 850	30.1	362 800	23.6	448 500	25.1	560 900	101.1
Daily average consumption (million gallons).....	17.500	71.4	30.000	18.6	35.572	81.3	64.504	268.6
Fixtures *per capita*.	0.523	33.3	0.697	26.5	0.882	32.1	1.165	122.8
Consumption *per capita*.....	62.758	31.7	82.7	— 4.1	79.31	45.0	115.00	83.3
Consumption per fixture	120.017	—1.1	118.7	—24.3	89.9	9.9	98.8	.....

One or two facts will illustrate this point:

In Boston the consumption per inhabitant for six of the more important industrial uses in 1880 and 1890 is shown in Table No. 5, all of the water for these uses being metered:

TABLE No. 5.

	1880. Gallons.	1890. Gallons.
Steam railroads.....	1.78	4.56
Sugar refineries.....	0.74	1.13
Gas-works.....	0.31	0.33
Electric light and power.....	....	0.97
Breweries.....	0.54	1.22
Elevators.....	0.95	2.05
	4.32	10.26

This increase, of about 6 galls. *per capita* for a few purposes, is nearly one-fifth of the total *per capita* increase for the same period.

Again, if from the fifty cities of the United States having the largest population are selected those where more than 40% of the taps were metered in the year 1900, it is found that all but two of them show an increase in consumption *per capita* during the past ten years, although the use of meters has been largely increased.

Mr. Brackett. The speaker is of the opinion that the best method of preventing waste on the premises of the water taker is by applying water meters and charging water takers for the actual quantity used. The time may come when the suggestion of Mr. Kuichling, that water may not be supplied under any condition for certain purposes, may be carried out, but, so long as the present enormous waste continues, the speaker believes that the payment for water used, as determined by meter measurement, will sufficiently restrict the waste.

Although engineers and water-works officials are now almost, if not quite, unanimously in favor of furnishing water by meter measurement, the water takers in many cities have yet to learn that it is for their benefit to pay for the actual quantity of water used, and it is the province of the engineer to educate public opinion and to encourage he gradual introduction of meters, rather than to attempt to force their use upon an unwilling public.

TABLE No. 6.

	Percentage of Taps Metered.		Gallons per Capita Consumption.	
	1890.	1900	1890.	1900.
Fall River	74.6	94.3	31	36
Worcester	89.4	94.3	59	70
Atlanta	89.6	91.6	36	84
Providence	62.8	82.6	48	54
Milwaukee	31.9	67.6	110	80
Toledo	9.4	50.3	72	119
Lowell	22.9	52.5	66	85
Syracuse	14.6	47.3	68	102
Dayton	3.8	46.7	47	62
Nashville	0.8	41.5	146	140
Averages	40.0	66.8	68	88

Mr. Christian. G. L. Christian, Assoc. M. Am. Soc. C. E. (by letter).—For several years the writer resided in a house where the water was metered. The house contained nine persons, five of whom were adults and four were small children. The house had all the modern improvements, and stood on a lot 55 ft. wide and 100 ft. deep. There was a sewer in the street, and the water pressure was about 50 lbs. per square inch. All the laundry work was done in the house, and a hose was used to water the garden and lawn when necessary.

In the mornings, before any water was drawn for use, it was allowed to run long enough to allow the escape of the water which had stood in the house pipes all night, and in hot weather, before drawing any for drinking purposes, it was allowed to run until it was as cool as it would get. All the water needed was used, without considering

Mr. Christian.

the cost, as the water bill was paid by the owner of the house. Although it was used lavishly when needed, waste was curtailed to a minimum by having the plumbing fixtures always in good order, and not allowing the water to run all night in cold weather to avoid freezing, that not being necessary, as the plumbing was well protected and the house well built.

The writer became interested in the matter of the water consumed in the house, and, upon examining the records in the office of the Board of Water Commissioners, was surprised to learn that at no time during the previous three years did the consumption exceed 17 galls. *per capita.*

Substantially, all the water consumed in Yonkers, N. Y., is metered, and, according to the annual report of the Board of Water Commissioners for the year 1897, there passed through all the meters, for domestic use, 278 984 470 galls., or 22.07% of the water pumped, as measured by plunger displacement; which, divided by 34 000 —the estimated population using the water—gives 22.5 galls. *per capita.*

During the same year there passed through manufacturers' meters 26.83% of the water pumped—equal to about 27.5 galls. *per capita*—making 48.9% accounted for.

The annual report of the same Board, for the year 1900, with 45 000 estimated consumers, shows a *per capita* consumption of 25 galls. as passing through all meters, except those of the manufacturers, and the amount passing through the latter was equal to a consumption of 13.5 galls. *per capita;* the total amount passing through all meters being 48.12% of the amount pumped. The other 51.88% is partly accounted for by street sprinkling, construction work, flushing sewers, fire purposes, the unreliability of the method of measuring —which probably records a little more than is actually pumped— and last, but not least, loss from leaks in the mains and service pipes.

The city is quite a manufacturing center, there being employed in its industries probably about 8 000 operatives, of whom perhaps 25% are skilled mechanics.

It is the home of many people of wealth, and has many miles of beautiful streets lined with handsome residences and well-kept lawns, and a general air of prosperity pervades the city as a whole.

When it is more economical to restrict waste than to build new works or enlarge those already built, the writer agrees with Mr. Herschel that all water used should be measured—not only when it begins its journey through the principal main, but through its various travels—by districting the city and metering the water entering each district; and then, with a meter on every tap, the location of leaks could be detected, and those who used or wasted the water would pay for it.

Mr. Christian. It would seem that a supply furnishing 60 galls. *per capita* should be ample for municipalities which have no manufactures of moment, providing good plumbing is required, and no great amount of leakage is allowed in the mains or service pipes.

Mr. Crowell. FOSTER CROWELL, M. Am. Soc. C. E. (by letter).—The writer is greatly interested in the subject of the proper allowance for consumption of water and the adoption of practical measures to reduce waste in public water supplies. With Mr. Desmond FitzGerald, he believes in taking every reasonable step to stop needless waste. His own investigations lead him to endorse thoroughly what Mr. Trautwine has stated, both as regards the enormous extent of waste now going on in our principal cities and the apparent hopelessness of reaching the public ear and mind in the effort to educate the community in this respect.

In 1900 the Engineering Committee of the Merchants' Association of New York, of which the late Thomas C. Clarke, Past-President, Am. Soc. C. E., was Chairman, requested the writer to investigate and report upon the use and waste of water in the Boroughs of Manhattan and the Bronx in New York City.* In pursuit of that duty he had sought and obtained many facts relating to the supplies of water actually consumed in various cities, a comparison of which convinced him of the fallacy of the reasoning that is so often heard, that because a city is rich and prosperous, therefore it will, on that account, naturally and properly need a larger *per capita* supply than smaller places.

It is true that among individual consumers it is almost invariably found that in the houses of the wealthier class there is a very much greater use than in the houses of the poorer class, and that, as the standard of living is raised from decade to decade more or less in every class of people, the legitimate average *per capita* water consumption also increases; yet he differs from the general conclusion, believing that, of the class using water lavishly, the proportion may actually be smaller in the large city than in the small manufacturing town. An unsuccessful effort has been made to determine the actual proportions of wage earners to total population in several cities, for comparison. The figures of the United States census of 1900, which might throw light on this contention, are not yet available, and those of 1890 are of little service at this date, especially as a form of classification which would bring out clearly the point in question has not been used.

In the absence of statistics we can only fall back upon conjecture aided by analogy, and as New York, for instance, is the largest, or one of the largest, manufacturing centers in the world, is said to

*"An Inquiry into the Conditions Relating to the Water Supply of the City of New York," by the Merchants' Association of New York, August, 1900, page 187, "The Use and Waste of Water in New York City," by Foster Crowell, M. Am. Soc. C. E.

contain more wage earners and to have a far denser population than any other city of civilization, it cannot be admitted that it is a city of wealth and luxury, in the sense in which the term has been used in this discussion. Mr. Crowell.

The same is true, in a measure, of Philadelphia, though not to the same extent, so that it would be reasonable to expect a somewhat higher average *per capita* consumption there than in New York, while in Washington we should expect to find a much higher figure. The comparisons in the report referred to, containing the evidence from Fall River, Woonsocket, etc., with the observed facts relating to water consumption in New York and Philadelphia, to which use of such figures Mr. Hering appears to object, were for the purpose of directing attention to the enormous difference in the quantities of water consumed in metered and unmetered cities, as illustrated by those cases. Here, it is important to revert to the fact that lavish use is not necessarily waste, and this should be emphasized, because the confusion between these terms does not appear to be confined to laymen, but seems to exist also in the quarter where we should least expect it, the minds of many engineers.

It is also well to understand clearly what is meant by the term "*per capita*," which is only used here because it is a term of convenience. It is used variously, and generally loosely. Here, it is meant to indicate the proportional part of the water supply that is represented by each individual in the entire population served with water from a given supply, and including all the water used, for whatever purposes.

The writer takes exception to Mr. Hering's proposition, that meters should be omitted in houses for small families on the ground of economy. Because it may cost less to let a certain amount of water go to waste than to provide means for checking it is no argument when the community is largely made up of small families, especially as there is no likelihood that under such conditions, either the wastage or the use would be limited to the certain amount. In the writer's judgment the use of meters should be universal and uniform; they should be owned by the city, or water company, in all cases. In cases where the supply is by the city, a certain small fixed allowance should not be charged for, though measured and taken account of. Beyond that, the meter charges should apply to all water consumed, on a rate schedule decreasing as the quantity consumed increases.

The writer would join with others in their protest against that ancient fraud, the dictum that "water should be free as air." It cannot, in the nature of things, be made free, *ergo* its cost should be reduced to the minimum. The way to do that is to make the supply go as far as possible.

Even where the public water supply is copious and need not be

Mr. Crowell. stinted, the wasting of it should not be tolerated. In the nursery we were taught that "wilful waste makes woeful want," and we should not lose sight of the fact that the less water we waste, the more we can use at the same cost, or the less that which we do use will cost us.

It makes no difference, in principle, whether wastage is due to defects in the distribution system or carelessness by users; in either case, or in both cases, it should be stopped. The latter case is the easier to deal with, for, upon the introduction of meters, as the writer has elsewhere stated:

"Every householder would thereupon become directly interested in the reduction of the waste, and the inherent trait of human nature which previously had led him to consider that the more water he used the more value he was getting back from his taxes, would now cause him to feel concerned lest he should suffer by being made to pay for water he did not use. That is the experience in other places where meters have been universally applied, and to a greater or less extent is to be counted upon."

Mr. FitzGerald's shining example is a case in point.

It is unfortunately true that in several prominent instances engineers of high authority, after pointing out the probability that tremendous waste was going on, lacked the courage of their convictions, and, instead of urging the adoption of radical means for its stoppage, advised that sources for additional supply should be provided, thereby increasing the opportunity for waste.

Undoubtedly, the legitimate uses of water will increase, both in quantity and in kind, faster than the increase of population. It would seem, therefore, right and prudent for every city to make some provision in advance for the supply of the future, even extending to the purchase of water-sheds and reservoir sites if need be, but further expenditure for new water-works is not justifiable until the point is reached where the legitimate use approaches the working supply, with a reasonable margin of time for their preparation.

Mr. Sherman. CHARLES W. SHERMAN, Assoc. M. Am. Soc. C. E. (by letter).—The writer has recently compiled a series of tables,* containing more or less complete statistics relating to the water-works of sixty-one cities and towns, for the year 1900. From these statistics Table No. 7 has been prepared. This table gives statistics relating to the consumption of water and use of meters in forty cities and towns, these being all for which sufficient data of this kind were available.

A part of this information is reproduced in the diagram, Fig. 4, which shows for each town the relation between the *per capita* consumption of water (per consumer where given, otherwise per inhabitant) and the percentage of services metered.

* Published in the *Journal* of the New England Water-Works Association, for September, 1901.

Mr. Sherman.

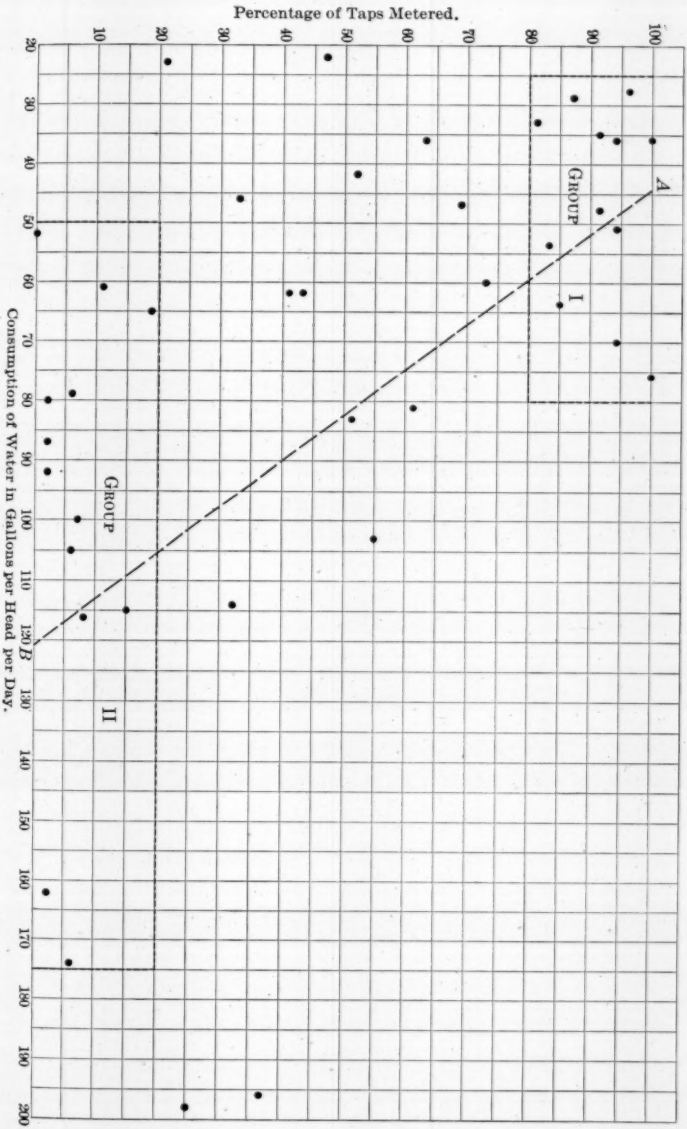

Fig. 4.

Mr. Sherman. From the forty points on the diagram two groups have been selected. Group I includes all the places, twelve in number, having more than 80% of the services metered. It will be noted that the range of consumption for this group is from 28 to 76, or 48 galls. per head per day. Group II includes the 13 places having less than 20% of their taps metered, and in this group the range of consumption is from 52 to 174, or 122 galls. per head per day. Taking, as mean consumption for each group, the average between the highest and lowest consumption in that group, and plotting these figures as corresponding to 90 and 10% of services metered, respectively, points are obtained through which the line *A B* has been drawn.

TABLE No. 7.—CONSUMPTION AND USE OF WATER IN FORTY AMERICAN CITIES AND TOWNS.

City or Town.	Population, 1900.	Consumers, 1900.	Average Consumption of Water, Gallons per Day.		Percentage of total consumption metered.	Percentage of taps metered.	Percentage of receipts from metered water.
			Per inhabitant.	Per consumer.			
Andover, Mass.	6 813	.....	60	.....	.....	73	.....
Bay City, Mich.	28 000	16 000	112	196	21	37	.....
Billerica, Mass.	2 780	1 230	20	46	.....	33	19
Brockton, Mass.	40 063	35 000	29	33	51	81	87
Burlington, Vt.	18 800	18 100	46	47	53	69	77
Cambridge, Mass.	91 886	.....	79	.....	.....	6	.....
Chelsea, Mass.	34 000	34 000	87	87	12	2	.....
Erie, Pa.	52 733	.....	162	.....	.....	2	18
Fall River, Mass.	107 623	104 523	35	36	.....	94	.....
Fitchburg, Mass.	31 531	27 000	.....	103	.....	55	67
Geneva, N. Y.	11 000	.....	81	.....	24	61	.....
Haverhill, Mass.	37 175	.....	61	.....	.....	11	.....
Holyoke, Mass.	46 204	45 204	103	105	14	6	17
Ipswich, Mass.	4 658	.....	23	.....	.....	21	.....
Keene, N. H.	9 300	8 000	150	174	.....	6	.....
Lowell, Mass.	95 000	.....	83	.....	.....	51	.....
Lynn, Mass.	73 600	72 128	.....	65	20	19	.....
Madison, Wis.	19 164	16 548	44	51	.....	94	.....
Marlboro, Mass.	13 609	11 726	36	42	.....	52	.....
Metropolitan W. W., Mass.	815 400	.....	116	.....	.....	8	.....
Middleboro, Mass.	.....	3 750	56	62	48	43	66
Minneapolis, Minn.	202 718	95 000	93	198	19	25	.....
Nantucket, Mass.	3 002	2 400	42	52	.....	0	.....
New Bedford, Mass.	62 500	55 000	101	115	23	15	.....
New London, Conn.	17 500	15 800	91	100	.....	7	.....
Newton, Mass.	33 587	32 800	62	64	55	85	86
Oberlin, Ohio.	4 800	2 800	21	36	43	68	.....
Providence, R. I.	187 297	187 297	54	54	60	88	.....
Reading, Mass.	4 969	4 200	30	35	.....	91	.....
Springfield, Mass.	62 059	48 200	88	114	21	32	31
Taunton, Mass.	31 006	26 116	53	62	37	41	.....
Waltham, Mass.	23 700	23 250	90	92	5	2	.....
Ware, Mass.	8 263	.....	36	.....	56	100	79
Wellesley, Mass.	5 072	4 929	47	48	53	91	.....
Whitman, Mass.	6 172	.....	22	.....	.....	47	63
Winchendon, Mass.	5 001	.....	12	28	47	96	.....
Woburn, Mass.	14 254	.....	80	.....	10	2	.....
Woonsocket, R. I.	32 500	32 000	28	29	.....	87	.....
Worcester, Mass.	118 421	113 217	.....	70	52	94	.....
Yonkers, N. Y.	.....	.....	76	.....	48	100	.....

Mr. Sherman.

Judging from these figures alone, it would seem that this line marks a fair limit, beyond which consumption (including waste) may justly be considered excessive. This line would give, for normal consumption, 44.5 galls. per head daily with all services metered, and 121 galls. where no meters are used. Of course, generalizations of this kind are of very limited application, and must be used only with the greatest caution.

The statistics of Woonsocket, Fall River, Brockton, and other places are frequently quoted as showing that a consumption (including waste) in excess of 35 galls. per head daily is excessive, and, by the use of meters, could, perhaps, be reduced to that amount. The writer believes that argument to be ill chosen, as all the cities named are manufacturing centers, containing a large percentage of operatives' houses or tenements, in which there are few fixtures and in which the use of water is restricted. In his opinion, the smallest quantity of water which will be consumed in dwellings of a high class, where the plumbing is carefully looked after and leaks are promptly repaired, will be, generally, not less than 35 galls. per head per day, and the average quantity used in such houses will be more nearly 45 galls. per head per day. The average domestic use in a city containing all classes of houses in ordinary proportions would then probably not be less than 30 galls. per head per day, to which must be added the quantity used for fires and other public purposes and the leaks in mains and services, in order to obtain the total consumption (including waste).

This opinion regarding the legitimate use of water is based on actual observations. For the past year the writer has taken readings of the water meter in his house, at intervals of about a week, and has taken great care to have no leaky fixtures and to reduce as far as feasible the unreasonable drawing of water, without curtailing its proper use. His records show the average use of water in that time to be at the rate of 31.5 galls. per head per day, the weekly average ranging from 21.2 in November to 47.8 galls. in July. A friend, similarly situated, has furnished records showing that in his house, where the use of water is somewhat lavish, but where leaks are not tolerated, the consumption has averaged 47.2 galls. per head per day for the past three years, the yearly averages being 49.5, 45.4 and 46.7 galls., respectively.

Other interesting deductions can be made from such data as are given in Table No. 7. For instance, if the percentage of receipts from metered water is very nearly the same as the percentage of taps metered, the inference would be that the unmetered consumers were not making an excessive use of water, and that little, if any, reduction of consumption (including waste) would result from extending the use of meters. In case, however, a large percentage of the total consumption is used by a few consumers, through meters, this deduction may

Mr. Sherman. not be warranted. Again, in cases where 90 to 100% of the taps are metered, if only 50 or 60% of the consumption is accounted for by meters, then the loss from leakage, or the public use of water, or both, must be very great.

Mr. Kuichling. E. KUICHLING, M. Am. Soc. C. E. (by letter).—Several points of interest have been brought out in this discussion, although the principal feature seems to be the unanimity with respect to the desirability of applying meters to service pipes. By this agreement a very important step in the advancement of water-works practice may be regarded as having been taken. There is, however, a noticeable tendency to consider the meter as the chief agency in preventing waste, and to regard legitimate use as "everything that the consumer chooses to pay for." The writer does not believe that this proposition is tenable without the vital qualification that such use shall always be reasonable; and the determination of reasonable allowance for different classes of consumers appears to him to be eminently desirable in the premises.

It has been said that shrewd manufacturers do not require to be taught how to get their water supply from the community for the least money, with the implication that they are the best judges of their own needs. While this may be true in some instances, it cannot be accepted as a general statement, inasmuch as enormous differences in consumption, in establishments of equal producing capacity, are encountered on every side, and the only coincidence of judgment seems to be to have a great abundance of water at hand. Demands for service pipes of unnecessary and preposterous magnitude are made so often that it is a conspicuous exception to meet an applicant who can justify his estimate. For this reason the water-works superintendent should know the necessities of the different consumers, and be able to adapt the size of the service pipes thereto in a rational manner. Especially is this the case when the water supply of a city has reached its limit, and it becomes imperative either to prevent useless consumption or to provide new works at large expense.

The exercise of sound knowledge of trade requirements in preventing waste cannot properly be characterized as "paternalism" in municipal government or water-works administration, and no officer should be deterred by the sight or sound of the objectionable phrase from acquiring and applying such knowledge for the ultimate benefit of the community or corporation he serves. By so doing he will make his position much more acceptable to the public, and far less mechanical to himself. The same remark is equally pertinent when meters are used, as the limitations of such measuring devices must be fully understood by an efficient officer. Several references to this point have been made in the discussion, and to emphasize it somewhat more strongly, the following instance in the writer's experience is cited.

Mr. Kuichling.

One of the largest water consumers in the City of Rochester, N. Y., obtained his supply through a 4-in. pipe and a 4-in. meter, and for several years the monthly consumption was fairly uniform. It then decreased abruptly in great degree, and on inquiring for the reason the answer was given that water was taken from another source. The meter was tested in place under relatively large flow, and found to give a registration of about 85% of the observed discharge into a capacious tank. Further investigation disclosed the fact that the employees of the consumer were still using the city water as freely as before, and that therefore the consumption should be about the same, also that the water was delivered into large tanks almost continuously in a stream of moderate size without appreciable registration. An accurate new meter was then substituted for the old one, with the result that the registration promptly returned to its former magnitude. On examining the old meter it was found that, owing to wear of parts, registration of discharge occurred only at large flows, as aforesaid.

The foregoing is only one of many similar cases, in the writer's experience, which indicate that the indiscriminate use of meters is not an infallible remedy for the prevention of waste, and that a water-works superintendent should possess a fairly good knowledge of a consumer's legitimate use of water, as well as of the various ills to which his own measuring devices are subject. In these directions there is still room for many valuable additions to our stock of information, and it is hoped that in future discussions such may be elicited.

By some writers, much stress is laid upon average figures of water consumption, apparently without reference to local trade or manufacturing conditions. Such figures are often illusory, however, when applied to a particular case, and, when covering a very large population, can be accepted only as a measure of domestic use. Thus, from Mr. Brackett's valuable contribution, it is learned that while the total average daily consumption *per capita* in Boston in 1880 was 82.7 galls., the use of water by six of the more important industries in that city amounted to only 4.3 galls. *per capita* during the same year; and that in 1890 the figures were 79.3 and 10.3 galls. *per capita*, respectively. Similar low proportions are found in other large manufacturing cities, although in some smaller towns the ratio is much higher. With the more extensive use of meters in our country, such data are gradually becoming available, and in time we may look forward to a much better presentation of this part of the subject than is now possible.

Those who are interested in average values of *per capita* water consumption will find some useful material in a table compiled by Allen Hazen, M. Am. Soc. C. E., from a "Report of the London County Council."* This table gives the population and use of water in 36

* Published in *Engineering News*, February 16th, 1899, p. 111.

Mr. Kuichling. English provincial municipalities for the years 1896 and 1897, but does not include London. The figures of consumption are all expressed in United States gallons.

Arranged according to population, the list would begin with Manchester, having a population of 899 093, and an average daily water consumption of 40 galls. per head, and end with Gloucester, having a population of 43 000, and an average daily consumption of 21 galls. per head. According to consumption, Middlesborough would head the list with an average of 61 galls. per head, while Wigan would end it with only 20 galls. per head per day. The total population in the 36 cities considered is 7 145 669, and the aggregate daily consumption is 238 174 154 galls., giving an average of 33 galls. per head per day.

A somewhat more extensive compilation* of similar character was made by the writer three years ago. All data were obtained from the best available sources at the time. The general results obtained are as follows:

In 100 American cities of 30 000 inhabitants and upward, representing a total population of 16 582 100, the average daily consumption was 112 galls. *per capita;* in cities of 1 000 000 inhabitants and over, it was 116 galls.; in cities of from 600 000 to 300 000 inhabitants, it was 122 galls.; in cities of from 300 000 to 100 000 inhabitants, it was 106 galls.; in cities of from 100 000 to 50 000 inhabitants, it was 105 galls.; and in cities of from 50 000 to 30 000 inhabitants, it was also 105 galls. The total quantity of water consumed per day by said 100 cities was 1 863 248 300 galls.

In 43 English, Scotch and Irish cities, ranging from 2 800 to 4 263 300 inhabitants, representing a total population of 9 616 400, the average daily consumption was 37.3 galls. *per capita.*

In 66 German cities, ranging from 16 400 to 1 122 300 inhabitants, representing a total population of 5 806 300, the average daily consumption was 21.2 galls. *per capita* in 1884, while in 1896 it was 23.8 galls. for an aggregate population of 3 352 400 in 14 large cities.

In 20 French cities, ranging from 4 130 to 2 269 023 inhabitants, representing a total population of 3 677 286, the average daily consumption was 59 galls. *per capita* in 1887.

In 12 Italian cities, ranging from 24 200 to 530 900 inhabitants, representing a total population of 2 533 600, the average daily consumption was 67.6 galls. *per capita* in 1892.

In 23 Spanish, Belgian, Dutch, Scandinavian, Russian, Austrian, Swiss and other European cities, ranging from 23 300 to 929 100 inhabitants, with an aggregate population of 5 590 300, the average daily consumption was 23.8 galls. in 1883.

In 12 Indian, Australian, Egyptian and South American cities,

* Published in the *Transactions* of the Association of Civil Engineers of Cornell University, for 1898.

ranging from 30 000 to 773 200 inhabitants, representing a total population of 2 884 900, the average daily consumption was 23.4 galls. *per capita* in 1883. Mr. Kuichling.

TABLE No. 8.—Statistics of Water Supply of Forty-three German Cities of over 50 000 Inhabitants. Abstracted from "Statistische Zusammenstellung der Betriebsergebnisse von Wasserwerke," Cologne, 1899.

Number.	City.	For fiscal year.	Estimated population.	Average daily consumption, U. S. gallons, per day.	Percentage of Average Daily Consumption.			Consumption, in U.S. Gallons, per Head, per Day.	
					Metered.	Estimated.	Not accounted for.	Maximum.	Average.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	I.—Supply Obtained by Pumping.								
1	Berlin	1896-7	1 763 049	36 300 000	84.6	4.2	11.2	30.4	20.6
2	Gelsenkirchen and vicinity	1897	573 000	16 600 000	93.4	4.4	2.2	.....	28.9
3	Charlottenburg and vicinity	1897-8	430 320	7 270 000	85.1	.....	.....	34.0	17.0
4	Dresden	1897	352 700	8 750 000	65.1	34.8	0.1	39.2	24.9
5	Hanover	1897-8	272 000	4 640 000	74.6	8.4	17.0	*32.3	*22.7
6	Magdeburg	1896-7	222 238	5 690 000	82.6	5.4	12.0	33.0	25.6
7	Dusseldorf	1897-8	205 300	4 960 000	66.9	28.1	5.0	38.4	24.0
8	Königsberg	1897-8	178 000	3 130 000	50.0	49.9	0.1	29.6	17.4
9	Stuttgart	1897-8	166 798	4 120 000	38.6	42.0	9.4	44.0	24.6
10	Elberfeld	1897-8	158 000	6 270 000	64.4	30.4	5.2	51.2	39.7
11	Stettin	1897-8	150 000	2 800 000	84.0	2.3	13.7	28.5	18.7
12	Zurich	1896-7	145 000	6 690 000	48.4	46.2	5.4	68.0	46.0
13	Aachen	1897-8	138 523	2 540 000	75.6	3.2	21.2	24.3	18.2
14	Barmen	1897-8	138 000	6 280 000	32.0	58.0	10.0	63.5	45.3
15	Halle	1897-8	123 500	2 620 000	80.5	2.5	17.0	31.4	21.2
16	Strassburg	1897-8	117 500	2 490 000	52.8	27.7	9.5	38.5	21.2
17	Krefeld	1897-8	108 253	3 360 000	25.7	52.3	22.0	51.2	30.9
18	Brunn	1897-8	102 509	3 270 000	31.5	56.5	12.0	45.5	31.7
19	Mannheim	1897	100 000	2 090 000	72.7	18.1	9.2	36.2	20.9
20	Kiel	1897-8	90 000	1 570 000	68.2	20.9	10.9	25.4	17.5
21	Mulheim	1897	82 910	1 910 000	45.8	52.3	1.9	32.8	23.0
22	Geneva	1897	80 000	32 100 000	19.0	55.5	25.5	510.0	400.0
23	Erfurt	1897-8	80 723	1 110 000	80.0	13.8	6.2	.....	13.8
24	Mainz	1897-8	78 000	985 000	93.4	1.8	4.8	21.4	12.6
25	Posen	1897-8	75 000	986 000	79.6	0.1	20.3	20.6	13.2
26	Würzburg	1897	71 000	4 030 000	5.5	88.4	6.1	58.3	56.9
27	Bonn-Godesberg	1897	69 000	1 800 000	50.8	48.4	0.8	47.0	25.9
28	Pressburg	1897	64 000	552 000	90.0	0.0	10.0	14.3	8.7
29	Darmstadt	1897-8	63 900	1 315 000	86.3	7.9	5.8	36.7	20.3
30	Hagen	1897-8	63 000	2 650 000	36.8	53.6	9.6	56.9	42.0
31	Gladbach	1897-8	60 963	980 000	59.2	30.9	9.9	28.5	16.1
32	Münster	1897-8	59 700	1 590 000	56.0	34.0	10.0	49.2	26.7
33	Potsdam	1897-8	58 355	735 000	93.9	1.3	4.8	25.3	12.7
34	Rostock	1897-8	50 100	1 720 000	26.5	71.3	2.2	52.0	34.0
35	Furth	1897	50 000	698 000	72.8	15.0	12.2	23.8	14.0
	Totals and averages	.....	6 541 341	184 601 000	59.9	.....	.....	.....	28.2
	Totals and averages, exclusive of Charlottenburg	.....	6 111 021	177 331 000	58.9	29.5	11.6	.....	29.0
	Totals and averages, exclusive of Charlottenburg and Geneva	.....	6 031 021	145 231 000	67.7	23.8	8.5	.....	24.1

* Includes river-water supply.

Mr. Kuichling.

TABLE No. 8—(*Continued*).

Number.	City.	For fiscal year.	Estimated population.	Average daily consumption, U. S. gallons per day.	Percentage of Average Daily Consumption.			Consumption, in U.S. Gallons, per Head, per Day.	
					Metered.	Estimated.	Not accounted for	Maximum.	Average.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	II.—Supply Obtained by Gravity.								
36	Munich	1897	424 000	20 700 000	83.1	....	....	.....	48.6
37	Nürnberg	1897	177 000	3 640 000	62.5	17.9	19.6	29.5	20.2
38	Danzig	1897	135 000	2 850 000	50.5	28.3	21.2	.....	21.2
39	Basel	1897	96 000	3 790 000	75.2	1.5	23.3	55.0	39.4
40	Weisbaden	1897-8	77 000	1 930 000	68.9	26.9	4.2	38.0	24.9
41	Plauen	1897	60 000	513 000	84.8	12.9	2.3	.....	8.4
42	Freiburg	1897	55 500	3 930 000	35.1	51.9	13.0	78.0	71.0
43	Metz	1897-8	55 300	1 800 000	11.7	60.3	28.0	40.0	32.5
	Totals and averages, Nos. 36-43, inclusive	.....	1 079 800	39 153 000	69.3	....	....	.....	36.3
	Totals and averages, Nos. 37-43, inclusive	.....	655 800	18 453 000	53.7	28.4	17.9	.....	28.1
	Totals and averages, Nos. 1-43, inclusive	.....	7 621 141	223 754 000	61.6	....	....	.....	29.4
	Totals and averages, exclusive of Nos. 3 and 36	.....	6 766 821	195 784 000	58.5	29.3	12.2	.....	28.9
	Totals and averages, exclusive of Nos. 3, 22 and 36	.....	6 686 821	163 684 000	66.2	24.2	9.6	.....	24.5

In the 176 European and other foreign cities mentioned above, the total population was 30 108 800, and the average daily consumption was 35.6 galls. *per capita;* and combining these figures with those relating to the said 100 American cities, will give a total of 46 690 900 inhabitants, consuming on the average 62.8 galls. of water per head per day. The figures also indicate that the consumption in the American cities is about three times greater than that in the European cities.

With respect to the influence of meters in reducing consumption or waste, little need be added to the excellent statistical work performed by Messrs. George I. Bailey and Charles W. Sherman. To supplement the same, however, Table No. 8, relating to 43 German cities, wherein meters are used more or less extensively, is submitted as a matter of interest. This table was compiled by Allen Hazen, M. Am. Soc. C. E., and the writer, from the official "Report of the German Association of Water-Works Managers" for 1899, and needs no further explanation than is given by the column headings. It may also be added that the original statistics mentioned are in great detail, and their form is commended for use by similar associations in other parts of the world.

Mr. Kuichling.

According to Table No. 8 the average consumption in the 43 cities specified is 29.4 galls. per head per day for an aggregate population of 7 621 141, and of this quantity 61.6%, on the average, is metered. It is also very interesting to examine the figures given in Columns 7 and 8, relating to the percentages of the supply which are respectively estimated and not accounted for. In the former the range is from 0.0 to 88.4%, with an average of 29.3%; while in the latter the range is from 0.1 to 28.0%, with an average of 12.2% for 41 cities. Those who have had charge of the water supply of a city of 50 000 or more inhabitants in our own country will doubtless look with much suspicion upon the accuracy of so small an amount as 0.1% unaccounted for when 49.9% is estimated, as in the case of Königsberg—No. 8 in the list. Nevertheless, the figures taken collectively are instructive, inasmuch as they indicate that the officers in charge of German waterworks have adopted a closer system of accounting for the water distributed by them than their American brethren.

It may also be remarked that in many German cities manufacturers who use large quantities of water in their business generally provide themselves with an independent supply, so that the figures given in Table No. 8 relate mainly to the domestic consumption and the quantity used for general public purposes. Furthermore, the number of faucets and water fixtures in the buildings is generally much smaller than in the corresponding classes of American houses, whereby the waste or leakage is proportionally reduced.

The quantity not accounted for may perhaps be taken as the leakage of the distributing system and the "slip" of the meters. In this event, the very low percentages given in a number of cases are probably erroneous, as neither their meters nor their methods of laying pipes are materially better than our own; and if the loss in extensively metered American cities cannot be kept within a limit of 10%, it is fair to infer that a similar loss will prevail elsewhere. On this point, however, adequate data are still lacking, and it is hoped that such will be supplied by water-works officers in the near future.

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TRANSACTIONS.

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No. 909.

THE ANTECEDENTS OF THE SEPTIC TANK.

By Leonard Metcalf, Assoc. M. Am. Soc. C. E.

Presented September 25th, 1901.

WITH DISCUSSION.

In these days of rapid development in the science of bacteriology and its many practical applications to the arts, the subject of the bacterial purification or disposal of sewage has naturally arrested the attention of the engineer.

Certain well-defined principles are already recognized and generally accepted; for the rest, it is as is to be expected in any new science, for theories and misconceptions multiply as do the applications to new and untried conditions.

It is not, however, of the principles of the bacterial and septic tank treatment of sewage that the writer purposes to treat, but rather of its evolution and of the antecedents of the so-called septic tank in their bearing upon and relation to the collateral patent question and the very broad and far-reaching claims being made in certain cases under them. Indeed, the writer's attention was first drawn to this subject by the claims—in his judgment unwarranted—for infringe-

ment of patent rights in the construction of a sewage disposal plant designed by him for a large manufacturing company in New England. Since that time various claims for infringement in recently designed or constructed works have been made in different parts of the country, which has led the writer to present at this time a portion of the data gathered by him in his own studies, in the hope that it may be of some service to brother engineers.

Historical Sketch of Sewage Disposal Methods.

One is surprised to note of how comparatively recent origin are the various methods of sewage disposal other than the old privy and cesspool systems, which were frequently combined with agricultural disposal of the wastes. Indeed, most of the systems have been developed since the middle of the nineteenth century.

The privy was followed by the pail system and the cesspool; and then the cesspool from which the settled liquid was decanted to the sewers or upon land.

Next came the water-carriage system and the bitter war between the advocates of the *tout à l'égout*, and those advocating the primary separation of the solids before discharge into the sewer. About this time the Liernur system was also more or less in vogue in certain parts of the Continent, though it never gained much recognition in Great Britain.

Chemical precipitation was next developed, along, and in many cases combined, with sand filtration and broad irrigation; then storage and intermittent filtration, so carefully studied by the Massachusetts State Board of Health; and finally, within the last decade, the bacterial treatment of sewage, under which may be included the septic tank system; the septic tank system followed by intermittent filtration; the contact bed system—single, double or treble—and the various bacterial methods making use of the principles of bacterial contact in diverse ways, with or without such minor modifications as primary sedimentation, passing through roughing-filters or strainers, aeration, etc.

In London,* for instance, cesspools were in general use until 1847; thereafter, water-closets, with a sewerage system discharging directly

* "Public Health and Its Applications in Different European Countries," by Albert Palmberg, M.D., translated from the French by Arthur Newsholme, M.D. 1893. Also *Minutes of Proceedings*, Institution of Civil Engineers.

into the Thames, until 1859, when Bazalgette's plan was adopted and put under way. It was not until 1865, however, that the latter system was finished in its essential parts—comprising an intercepting sewer with storage tanks, discharging on the ebb-tide only—while the intercepting sewer on the north side of the Thames was not begun until ten years later.

So, too, in Paris, the water-carriage system, with final disposal of the sewage by irrigation, was very slow in gaining general favor. Probably nowhere was the cesspool or tank system so carefully studied and fully developed as at Paris, for which reason it may be of advantage here to allude to its various steps a little more fully.

In 1820, Paris* "made the fatal mistake of insisting, by ordinance, upon the use of cesspools," and for twenty years thereafter such sewers as were built received only the decanted liquid or overflow from these cesspools. As late as 1854, the Prefect of the Seine† presented a memoir suggesting two possible methods of sewage disposal for Paris. The first, for a system of sewers to carry the solids and liquids to a distant point for treatment; the second, for the retention of the solids and the discharge of the liquid only into the sewers. The latter method was adopted, and the execution of the plans was begun in 1867. "Hardly had the collectors and the network of sewers begun to spread, under the direction of Belgrand, than the *tinettes filtrantes* were put into use," say MM. Bechmann and Launay in their valuable paper on "L'État Actuel de l'Assainissement de Paris."‡

These various devices for separating the solid from the liquid portion of the wastes are interestingly described by M. A. Mille,§ the *fosses fixes*, from which the matter is raised by metallic vessels or vacuum pumps; the *fosses sèches*, used in the barracks; the *tinettes filtres* or *filtrantes*, which permit the liquids to escape, and reduce the refuse to one-fifth part; the *vidange automatique*, invented by M. Mouras to do away with the necessity for even periodic removal of the solids; and all those other devices for dividing or diluting the sewage, which did so much to delay the adoption of the water-carriage system.

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* *Minutes of Proceedings*, Institution of Civil Engineers, Vol. xxiv, p. 318. R. Rawlinson.

† "Assainissement des Égouts et des Habitations," par Eugéne Miotat, Architecte Expert, 1881, p. 14.

‡ *Annales des Ponts et Chausées*, Jan.-June, 1895, pp. 257-327.

§ "Assainissement des Villes par L'Eau, Les Égouts, Les Irrigations," par M. A. Mille; and "Fosses d'Aisances, Latrines, Urinoirs, et Vidanges. Historique, Construction," etc. F. Liger, 1875.

The very general use of these devices is indicated by the fact* that:

"There are (1878) about 70 000 houses in Paris, of which only 62 000 are registered as having some description of night-soil service. These comprise four systems:

	Number.
"*First.*—Water-tight cesspools built in masonry.....	86 000
Second.—Removable receptacles (*fosses mobiles ordinaires*), or wooden barrels.....	17 450
Third.—Movable receptacles (*appareils diviseurs*), in which the liquid portion only filters into the cesspools, having no connection with the sewer.....	10 770
Fourth.—Movable receptacles (*tinettes filtres*), in which the liquid portion only filters into the sewers.....	10 000
	124 220"

In 1880, however, in consequence of the experiments upon the purification of sewage, began at Clichy, by MM. Mille and A. Durande-Claye in 1868, and carried on by them with such signal success, and when the use of the *tinettes filtrantes* had been more or less proscribed, "the administration proposed the discharge of all sewage direct into the sewers, and the abolition of all fixed or movable cesspools."† This was adopted in the session of June 23d, 1880, and "the law contained in it the germs of the principles which were finally consecrated in the law of July 10th, 1884."

About this time the Technical Commission of 1882, on the Drainage of Paris, condemned the use of the various devices for separating the solid from the liquid portion of the sewage, and advocated the *tout à l'égout* system.

"The Commission did not admit that it was ever necessary to keep in the house a place where the products of daily life could ferment," and therefore expressed itself as seeing "in these last efforts, which aim to simplify the *fosses fixes*, the hypocrisy of the *tout à l'égout* without its advantages. They take nothing from the underground flow, and, when all is said, do not avoid the deposits in the cesspools."‡

* *Minutes of Proceedings*, Institution of Civil Engineers, Vol. liii, 1878, p. 201; "On the Main Drainage of Paris and the Utilisation of its Sewage," by Felix Target.

† "L'État Actuel de l'Assainissement de Paris," par MM. Bechmann et Launay pp. 257–327, in *Annales des Ponts et Chaussées*, Jan.–June, 1895.

‡ See also *Annales des Ponts et Chaussées*, July, 1885; "Commission Technique de l'Assainissement de Paris," par M. A. Durande-Claye.
See also "Assainissement des Villes par L'Eau, Les Égouts, et Irrigations," par M. A. Mille.

Nevertheless, these devices appear to have served their purpose economically and efficiently for a considerable period of years, as is indicated by the remarks of so able an engineer as M. A. Durande-Claye, who, in an address, before the International Congress of Hygiene at Geneva, upon "Vidanges et Égouts," said of the *fosses fixes*, "In spite of its imperfections and dangers, it was indeed a step in advance of the barbaric depots of antiquity and the middle ages."

On February 28th, 1887, a law was passed providing for turning all matter directly into the public sewers within certain limited districts, with certain restrictions.

In 1886 a commission was appointed to investigate foreign methods of sewage disposal, and on April 4th, 1889, was passed the act authorizing the purchase of land at Achères for irrigation purposes. Nothing definite was done, however, toward securing the direct discharge of household wastes into the sewers until the cholera epidemic of 1892, which again awakened public interest, and culminated in the passage of the act of July 10th, 1894. This act, say MM. Bechmann and Launay, "constitutes one of the most important acts in the history of Paris and the science of hygiene, and provided for the modern Paris sewage disposal plant."*

It will thus be seen that the water-carriage system and purification by irrigation were not adopted for general use in Paris until near the close of the last century.

To the student of septic tank sewage treatment, this long period of use and development of the *fosses* in Paris is full of significance and interest.

General Claims.

Turning now from the broad question of methods of sewage disposal, let us glance at some of the specific claims of novelty bearing upon the bacterial, and more particularly the so-called septic, treatment of sewage, recently advanced.

Claims are made for the discovery, or at least for novelty in the application, of the following principles :

Sedimentation; septic or anaerobic bacterial action and the conditions

* See *Annales des Ponts et Chaussées*, January, 1895, previously referred to.
Also "Les Travaux Souterrains de Paris," 1810-1878, par Eugène Belgrand, Vol. 5.
Also works of MM. A. Durande-Claye, and Eugène Miotat, previously referred to.
Also "Les Systèmes d'Evacuation des Eaux et Immondices d'une Ville," par C. von Overbeck de Meijer.

developing and fostering it; aerobic bacterial action; slow flow through long tanks and the greater uniformity in character of sewage resulting therefrom; alternate use of settling, decanting or bacterial tanks, or beds filled with certain materials; periods of contact or activity, rest and recuperation; rapid screening through coarse materials; aeration; and doubtless many others.

While, under the devices, novelty is claimed in the use of: Covered, as distinct from uncovered, tanks; artificial ventilation by chimney or otherwise; special shape and grouping of tanks; form of bottom; submerged inlet and outlet; barriers and baffle-boards; aerating weirs or steps; and many alternating devices for manipulating gates, regulating period of inflow, contact, discharge and aeration of the beds, etc.

Discussion of Claims.

Few engineers would dispute the statement that the bacterial purification of sewage is far better understood to-day than it was a generation ago; yet many—if not a majority—of the engineers who have studied the question, would be unwilling to allow the claim of novelty in the action which is produced in the septic tank. This action is essentially a natural one, a process of Nature, which it is certainly difficult, if not impossible, to prevent, under the normal conditions attending storage or sedimentation of sewage.

The essential difference between the old cesspool and the modern septic tank appears to consist in the period of contact of the fresh or stale sewage with the liquefying action of the anaerobic bacteria developed and contained in the tank.

In the first cesspools, this period of contact was determined by the frequency of emptying them; in the later ones, provided with an overflow, by their relative content as compared with the volume of sewage to be disposed of. The later cesspools, therefore—as, for instance, those consisting of a water-tight chamber discharging its overflow into a sewer, leaching cesspool or sub-surface irrigation system—did not differ essentially, in principle at least, from the modern septic tank; for, according to the present theory, a limited anaerobic action only is desired, sufficient to bring about the hydrolysis of the solids, fats and soaps by the bacteria or their enzymes, as Professor Kinnicutt has suggested.

Beyond this stage of liquefaction, it is apparently undesirable to go, for prolonged contact results in the formation of bacterial wastes or poisons, making the further purification of the sewage by subsequent nitrification more difficult, or, if subsequent purification is unnecessary, the effluent is at least much more offensive and perhaps likely to undergo secondary putrefaction when disposed of by dilution.

The bacterial liquefaction of the organic matter in sewage may not have been fully understood, but fermentation, maceration and bacterial putrefaction were well known upwards of fifteen or twenty years ago.

Thus the Mouras's Automatic Scavenger, hereafter referred to, was described by Abbé Moigno,* who believed that in the liquefaction of the organic matter, "The solvent action of sulphuretted hydrogen is called into play, and that a species of putrid fermentation is set up, which effects the liquefaction of the solid fæces," while the abstractor closes with the suggestion, "May not the unseen agents be those vibrions or anaerobies which, according to Pasteur, are destroyed by oxygen, and only manifest their activity in vessels from which the air is excluded ?"

In 1883 the late Edward S. Philbrick, M. Am. Soc. C. E., of Boston, in an interesting article in *The Engineering Record*,† upon the Disposal of Sewage by Sub-surface Irrigation, suggested the use of "a tank or tight cesspool in which the solid particles of the sewage may become macerated and finely divided by fermentation before entering the distribution pipes."

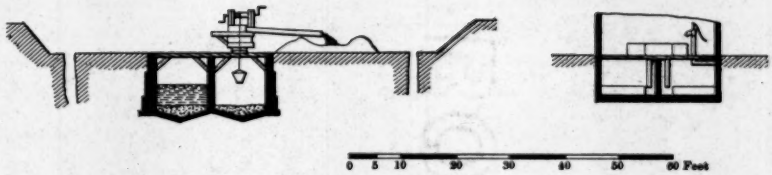
In 1887 Mr. Dibdin, in a paper on Sewage Precipitation delivered before the Institution of Civil Engineers, observed that:

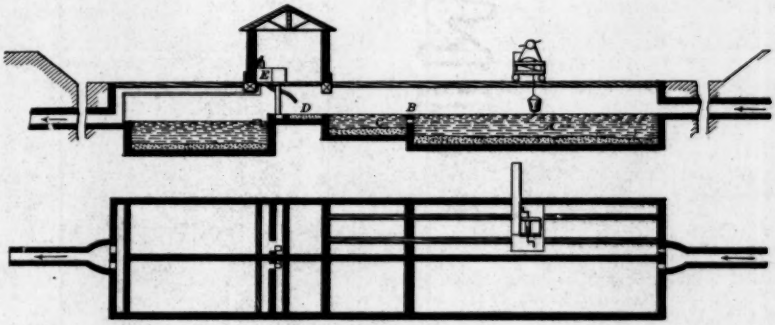
"One object claimed for the use of an excessive quantity of lime, and also for some other substances, is that they destroyed the living organized bodies, such as bacteria, etc., which give rise to the phenomena known as putrefaction. * * * As the very essence of sewage purification is the ultimate destruction, or resolution into other combinations, of the undesirable matters, it is evident that an antiseptic process is the very reverse of the object to be aimed at," and he further remarked that, "Very alkaline effluents, such as those produced by the use of lime in excessive quantities, are very liable to putrefy instead of becoming purified by oxidizing organisms."

As regards sedimentation by slow flow through long tanks, and the

* *Cosmos les Mondes*, December, 1881, and January, 1882.

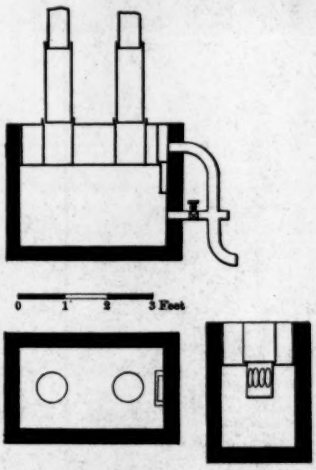
† *The Engineering Record*, May 10th, 1883, p. 530.





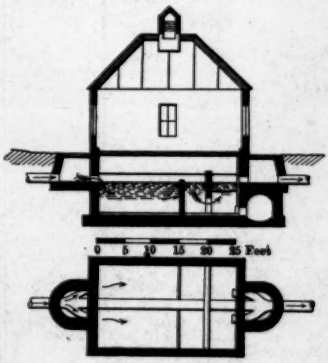
HENRY AUSTIN, 1852.

FIG. 1.



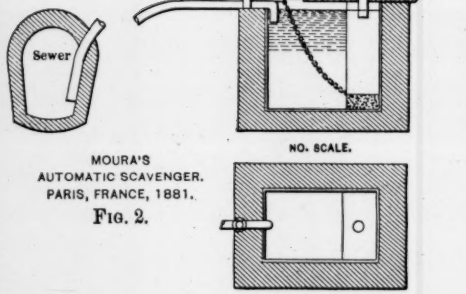
OF. L. PAGLIANI, ROME, ITALY, 1884, 1891.

FIG. 7.



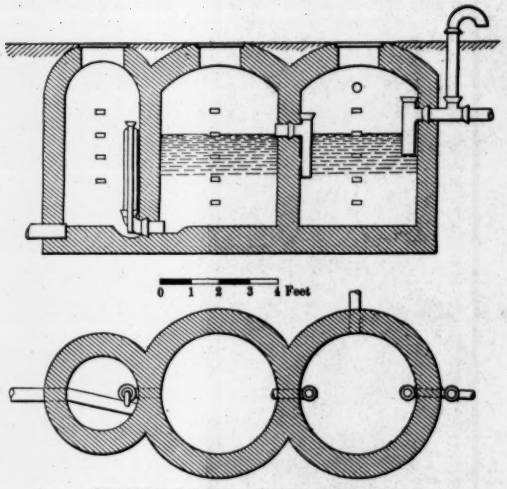
MARLBORO, MASS., 1892.

FIG. 8.



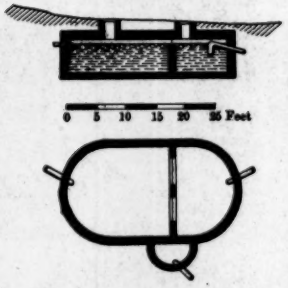
MOURA'S
AUTOMATIC SCAVENGER.
PARIS, FRANCE, 1881.

FIG. 2.



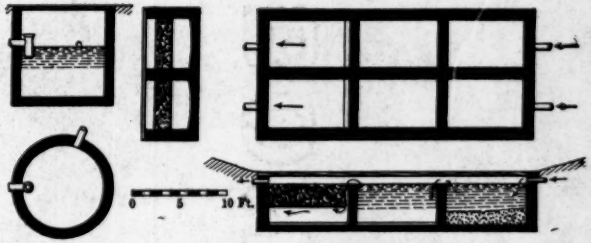
EDWARD S. PHILBRICK, BOSTON, MASS., 1883.

FIG. 4.



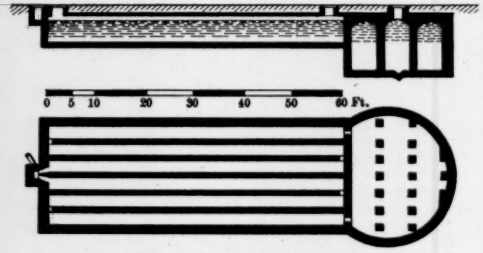
STATE HOSPITAL FOR THE INSANE.
WORCESTER, MASS., 1876.

FIG. 5.



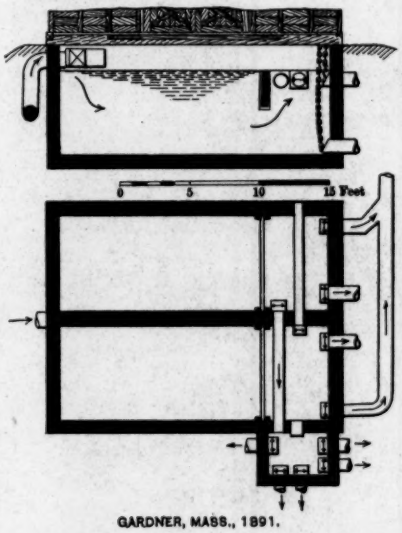
MEDFIELD, MASS., 1886.

FIG. 6.



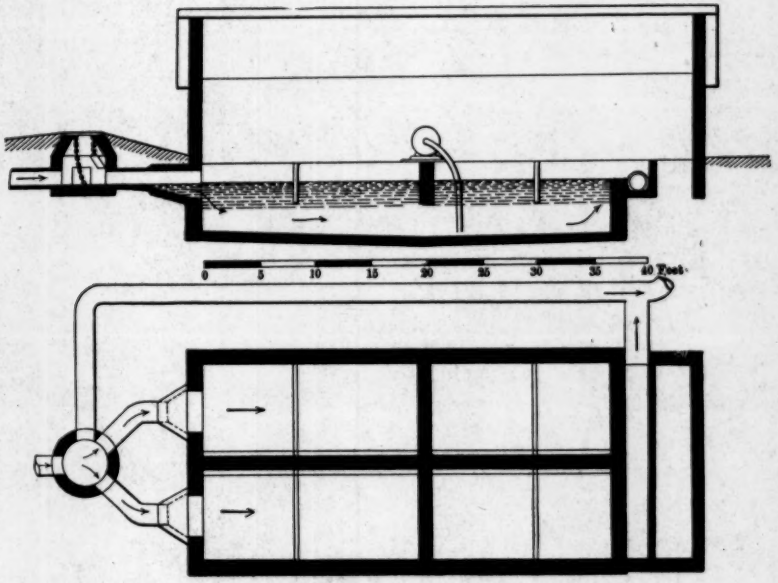
LAWRENCEVILLE, N.J. SCHOOL, 1882.

FIG. 3.



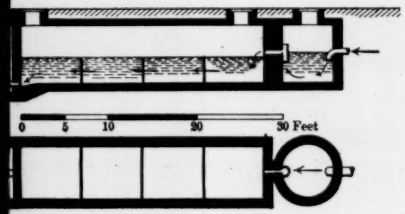
GARDNER, MASS., 1891.

FIG. 11.



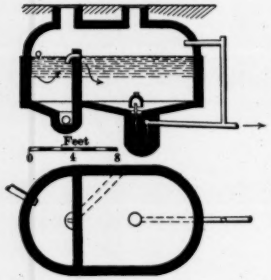
CHAMPAIGN, ILLINOIS, 1897.

FIG. 12.



ELLESLEY COLLEGE, WELLESLEY, MASS., 1892.

FIG. 9.



MASS., AGRICULTURAL COLLEGE, 1893.

FIG. 10.

greater uniformity of the sewage resulting therefrom, the use of baffle-boards, deflecting walls, etc., the principles involved have long been known and made use of—particularly in chemical precipitation systems.

Thus, taking several examples at random, Mr. Henry Austin,* in his interesting preliminary "Report of the Means of Deodorizing and Utilizing the Sewage of Towns," suggested means (see Plate XI, Fig. 1), for more effectually "separating the solid portion of the sewage from the water in which it is conveyed."

Professor H. Tanner, in his discussion of the valuable paper by Mr. Bazalgette upon "The Sewage Question," says:†

"* * * the engineer, by suitable mechanical arrangements, could go far in the direction of separating from the sewage the solid matter which it contained, and he could thereby greatly lessen the area of land which would otherwise be necessary. And here he would draw attention to the importance of constructing sewage tanks so as to facilitate the subsidence of the solid matter of the sewage, and to one modification which he had found successful. Tanks constructed for subsidence purposes should provide for the sewage passing through with the least possible agitation to the bulk of sewage which was there depositing its solid matter. By constructing a barrier across the tank, at the same level as the overflow at the opposite end, thereby forming a small preliminary compartment, the force of the current was kept within that portion of the tank, and the passage over such barrier displaced an equal volume quietly at the overflow outlet. This might be assisted by additional barriers, some raised from the bottom of the tanks, some pendant and sinking into the water for a depth of from 6 to 10 ins. Tanks thus constructed at the sewage works at Barking were fully equal to three times as much work as they had previously done. By these and similar arrangements which enabled the sewage to remain tranquil whilst subsidence was proceeding, it was possible, by mechanical arrangements, to accomplish much which would otherwise devolve upon the land. * * *."

And many other writers, as Mr. Philbrick, in 1883, have referred to the effect of tanks in "equalizing the flow of sewage."

Precedents abound for the use of roofs or covers, as well as of artificial means either for producing or preventing the free circulation of air in sewage tanks. Climatic conditions, proximity to dwellings, convenience, etc., are likely to be the controlling factors in their use

* London Sewer Reports, 1857, p. 79.

† *Minutes of Proceedings*, Institution Civil Engineers, Vol. xlix, p. 181, April, 1877, discussing C. N. Bazalgette's paper in *Minutes of Proceedings*, Institution of Civil Engineers, Vol. xlviii, pp. 143-248, Feb. 6th, 1877.

or disuse, as investigation has seemed to indicate that the bacterial (surface) mat is itself efficient as an excluder of light and air.

The value of baffle-boards and barriers, submerged inlets and outlets, in securing a slow and regular flow through, and preventing the formation of agitating currents in, sewage tanks, has already been referred to under the head of sedimentation, and some specific examples of their use are given in the following pages.

Space forbids an extended discussion of the validity of other claims mentioned, but, doubtless, certain of the devices for manipulating the beds and their appurtenances are novel in application, if not in form.

Illustrations and Precedents.

The following examples, arranged in chronological order, which have come to the writer in the course of his study of the subject, are given as illustrative merely of the principles alluded to, but without any idea of their being complete, or even perhaps the best examples on record. They have been drawn from text books and the many engineering journals within reach of the engineer, and published in this country, in England and in France. Time has, unfortunately, prevented a search of continental publications other than those of the French societies.

*Henry Austin's Parliamentary Report, 1852.**—In an interesting report upon the sewerage of towns, Mr. Henry Austin, of London, then Chief Superintending Inspector of the Board of Health, made some interesting recommendations for "An arrangement for separating the solid portion of the sewage from the matter in which it is conveyed, * * * with a view to the more efficient and economical working of the process;" that is, for primary sedimentation before treatment with lime and filtration.

"The great bulk of the solid matter, when the sewage comes to comparative rest in the reservoir, divides itself into two bodies; the heavier particles at once deposit themselves at the bottom, and the lighter portions collect, in a solid floating mass, on the surface. It appears to me that the chief proportion of these matters may be intercepted in the first tank, *A*, both above and below, without a filter, by a simple division, *B*, with basketwork and perforated boards in the

* "Report on the Means of Deodorizing and Utilizing the Sewage of Towns." Addressed to the Rt. Hon. The President of the General Board of Health, Hon. William Monsell, M. P., by Henry Austin, C. E., London, 1857. Presented to both Houses of Parliament.

middle, allowing the water to pass through only at a certain depth beneath the surface.

"The second tank or reservoir, *C*, need not then be so deep or so large as the first. At the end of it, filtering materials of different kinds should be arranged, through which the sewage would pass laterally; 1st, coarse screened gravel, beach or broken stones; 2d, gravel of medium coarseness; and 3d, some finer material. And there would be much advantage in having this filter, *D*, shallow and broad, rather than deep and narrow; the surface water only would pass away, and thus allow of further deposit from the main body before filtration; it would be subject to greater friction and disengagement of the solid matter and foul gases; and the filtering medium itself would be less readily clogged and would be more accessible for cleansing without removal, as it could be turned over in its place and subjected to complete washing, from a water tank, *E*, above, as often as desired. A very small area of filter so placed would be found sufficient for all practical purposes. * * *

"It is stated as one of the results of the Manchester experiments, that the lime falls at the rate of 1½ ins. per minute. The admixture being made as the sewage flows into the reservoir at each side. I propose that it should be intercepted by a fender, that it may not disturb the main body of water, and that the discharge to the outfall should take place over a weir, allowing only a surface film of the water, from which the lime has descended, to pass over.

"The depth of this surface water passing over, and the length of the reservoir, must be, of course, determined by the quantity which has to be discharged.

"The construction shown on the plans would be sufficient for a town of about 3 000 inhabitants, and would probably cost from £250 to £300, exclusive of the excavation of the site and the fencing shown, which would depend upon locality."

School Cesspool, Derbyshire, 1858.—Dr. Samuel Rideal, in the Cantor Lectures,* on the "Bacterial Purification of Sewage," delivered in 1899, mentions the time and skill spent by architects and others upon the design of cesspools in England in the early Victorian period, when sanitary progress first drew attention to its importance; and he refers to the construction in 1858 of a very large cemented brick pit for a school in Derbyshire, into which passed continuously the sewage of 250 to 300 persons, and the drainage from a farm. Whenever the pit was full its contents were pumped out from a point near the bottom and discharged by pipes over cultivated slopes. The mixed water was said to be clear and bright, except for an occasional turbidity from

Journal, Society of Arts, London, January 16th, 1899, p. 688.

the clay. At the periods of emptying, no nuisance occurred, though sometimes a faint odor was noticed when the wind was in the right direction.

*Settling Tank at Hospital for the Insane, Worcester, Mass., 1876.**—This settling tank (shown in Fig. 5, Plate XI), designed for an ultimate capacity of 600 persons, is of particular interest, as making use of the submerged inlet and outlet, diaphragm wall with punched screens and screen weirs, and the masonry roof. It forms a part of the broad irrigation system in use there.

Mouras' Vidange Automatique, 1881.—Mouras' Automatic Scavenger has been so frequently referred to and discussed in recent engineering literature as to make it unnecessary here to do more than give a few references† to it and briefly outline its action.

The scavenger was described by Abbé Moigno, editor of the *Cosmos les Mondes* as being a metal receptacle, hermetically sealed, continuous in working, absolutely inodorous, which rapidly transforms all it receives into a homogeneous fluid, only slightly turbid, free from deposit of any kind, and almost devoid of smell. Its general arrangement is shown in Plate XI, Fig. 2, and needs no further comment.

An interesting account of the application of the scavenger is given in the *Minutes of Proceedings* of the Institute of Civil Engineers, Vol. LXVIII, page 502, 1883–84. According to this, the scavenger was placed at the works of Mr. Herzog at Logelbach in the middle of the year 1883. It had a capacity of about 16 800 U. S. galls., and was used for three water-closets frequented by 150 people, the inflow being fairly constant in volume. The inlet and outlet pipes were submerged, of the form prescribed by the inventor, by which the vessel was hermetically sealed and the phenomena of oxidation practically prevented. The author states that complete putrefaction took place, by which all solid matters were dissolved in the space of thirty days.

Professor Rideal, commenting upon the size of the tank, says:

"It is not so large as to be impossible with ordinary sewages, but

* Rafter and Baker's "Sewage Disposal in the United States," p. 456.

† *Cosmos les Mondes*, December, 1881, January, 1882, and January, 1883.
Minutes of Proceedings, Institution of Civil Engineers. Foreign Abstracts: Vol. lxviii, pp. 350–352, 1881–82; Vol. lxxii, p. 359, 1882–83; Vol. lxviii, p. 502. 1883–84.
"Assainissement des Villes par l'Eau, les Egouts, les Irrigations," par M. A. Mille.
"Sewage and the Bacterial Purification of Sewage." By Dr. Samuel Rideal, p. 165.
Journal, Society of Arts, London. p. 710. 1899. Abstract by Rudolph Hering, M. Am. Soc. C. E.
Journal, Association of Engineering Societies, Vol. xxiv, February, 1900. Article on the Septic Tank, by Professor Leonard P. Kinnicutt.
Engineering News, April 15th, 1882, and reprint of the same, accompanied with letter of Kenneth Allen, M. Am. Soc. C. E., May 12th, 1898.

the fact that the effluent from such a tank was not sufficiently purified without further nitrification, prevented the automatic scavenger from being more generally adopted."

It appears to have been used for some years, however, in Paris, until the passage of the act requiring the discharge of all sewage direct into the mains, without preliminary treatment of any kind.

Mouras' scavenger is also described in a letter* by Scott Moncreiff, in which he says, referring to the later septic tank of Donald Cameron:

"It is nothing new, and has been in use by Beazley and Burrows, Architects, of Victoria Street, Westminster, for the last seven or eight years, in their ordinary practice of draining country houses."

School for Boys, Lawrenceville, New Jersey, 1882.†—A double receiving and settling tank (Fig. 3, Plate XI) for use at Lawrenceville, N. J., in conjunction with a sub-surface irrigation system, was designed in 1882 by J. James R. Croes, President, Am. Soc. C. E. Its capacity was planned for 400 persons, though it was used by only 180 for some time. The sewage at first amounted to about 6 000 galls. per 24 hours, though later, in 1893, it reached 20 000 galls. per 24 hours. The joint capacity of the two chambers amounted to about 11 000 U. S. galls. The sewage flowed in each septic tank down through one long central passage-way, back through the next, and out through the outside passage into the pump well, from which it was periodically discharged by a pulsometer pump into a sub-surface irrigation system. The inlets and outlets between the chambers were submerged, with a weir at the end of the last channel, over which the sewage fell in a thin sheet into the pump-well below. The chambers were ventilated by a flue to the boiler house.

This plant, built in 1882, embodied to a striking degree the very processes and devices now being used by others under claim of novelty, and covered by recent patents.

Massachusetts Reformatory, Concord, Mass., 1883.‡—A sewage disposal system, comprising sedimentation, storage and broad irrigation, was designed for this institution by William Wheeler, M. Am. Soc. C. E., in 1883. This system is of interest as illustrating the use of a small sedimentation and macerating chamber for the separation and liquefaction of the solids, submerged inlets and outlets, a large

* *Engineering* (London), September 25th, 1896, p. 407.

† Rafter and Baker's "Sewage Disposal in the United States," p. 511.

‡ See Massachusetts Public Documents, 1883, No. 13. Also Rafter and Baker's "Sewage Disposal in the United States," pp. 468-475.

storage chamber to contain approximately 14 hours' night flow of sewage, from which the liquid was pumped daily to a point approximately 1 ft. from the bottom. The tanks were also covered with a masonry roof, and provided with artificial ventilation to the chimney.

Edward S. Philbrick, 1883.—In an interesting article* on "The Disposal of Sewage by Sub-Surface Irrigation in Suburban Residences," Mr. Philbrick suggested the advisability of sedimentation, maceration and intermittent application of the sewage in sub-surface irrigation systems, and described a tank for accomplishing this purpose. Its construction is so clearly shown in Fig. 4, Plate XI, as to make further description unnecessary. Mr. Philbrick's discussion of the operation of the tank previously alluded to is excellent. The use of similar tanks has also been suggested in the more recent works of the late Colonel George E. Waring, Jr., and Mr. William Paul Gerhard.

In a later article† Mr. Philbrick, replying to an open letter asking about the dimensions for a flush-tank and settling basin for a hospital containing twenty-five beds, suggested the use of tanks 5 ft. in diameter by 5 ft. in depth, which would have given approximately 8 or 10 hours' sedimentation and septic action.

Medfield, Mass., 1886.‡—The "settling and filtering tanks" shown in Fig. 6, Plate XI, were designed for the Town of Medfield, Mass., by Eliot C. Clark, M. Am. Soc. C. E.§

Professor L. Pagliani, 1891.—At the VIIth International Congress of Hygiene and Demography, held in London in 1891, L. Pagliani, Professor of Hygiene, and Director of the Public Health, at Rome, Italy, presented a paper on "The Application of a Reservoir Interceptor, made on the Type of Mouras' Fosse, and of a Peat Filter for Sewage in the System of Separate Sewerage."

In this, he says: Considerations of economy made the *tout à l'égout* system impossible, and compelled the adoption of "a good dividing system," which should "separate the soluble or emulsionable materials of the privy * * * from the solid and heavy parts," and thus make

* *The Engineering Record*, May 10th, 1883.

† *The Engineering Record*, Vol. 8, p. 444.

‡ Rafter and Baker's Sewage Disposal in the United States, pp. 490-493. Also *Engineering News*, and *The Engineering Record*.

§ A detailed description of these tanks may be found in *The Engineering Record* May 10th, 1883.

possible the use of small-sized sewer pipes without danger of clogging. The tank is a type of Mouras' scavenger, made of cast iron or cement, as shown in Fig. 7, Plate XI.

"Experience has proved that such a reservoir can work properly without any inconvenience for a very long period, and perhaps indefinitely, when to it is supplied an abundant flowing of water; but if the water be in want, it may happen that the floating layer thickens so in the course of a few months as to make difficult the further flowing of the successive sewage. In this case, however, to put the apparatus to work again is sufficient to draw off the reservoir some liquid at one-fourth its height from the bottom by means of a pipe; the surface of the liquid with its floating solid layer is thus lowered," and the trouble remedied by further dilution. No deposit occurs in the pipe system.

"To filter the dissolved sewage that is drawn from the reservoir I resorted to peat with the best results."

He then describes his and Professor Monari's experiments with peat as a filtering medium, and mentions several plants of this character designed by him, notably in Rome, Turin and Massa.*

Gardner, Mass.; Marlboro, Mass., 1892.—Figs. 11 and 8, Plate XI, illustrate the tanks at Gardner and Marlboro, Mass., for description of which the reader is referred to Rafter and Baker's "Sewage Disposal in the United States."

Wellesley College, Mass., 1892, and Massachusetts Agricultural College, Amherst, Mass., 1893.—Figs. 9 and 10, Plate XI, are of interest chiefly as illustrating certain types of tanks. The former discharges intermittently into sub-surface irrigation pipes; the latter upon irrigation beds.

Champaign, Ill., 1897.—In Fig. 12, Plate XI, is shown the Champaign, Ill., tank, designed by C. N. Talbot, M. Am. Soc. C. E. Of it, Professor Talbot says:†

"This septic tank has been in operation since 1897, although it was designed in 1895. It was an outgrowth of the writer's experience with the Urbana septic tank, which he constructed in 1894. The ideas for this came from a study of various settling tanks connected with sewage disposal works, the action of large cesspools, and the 'automatic vault cleaner' of M. Mouras, described in *Engineering News* of April 15th, 1882. The Urbana tank was covered with

* Since making the above abstract of Professor Pagliani's work, the writer has noticed a short article in *The Engineering Record* of March 30th, 1901, upon "An Early Example of Combined Septic Treatment and Filtration of Sewage."

† *Engineering News*, August 17th, 1899, p. 112.

plank and earth, and had a submerged outlet. Observation on its action proved that a material purification of the sewage was made, and the efficiency of the tank has been quite satisfactory. It may be noted that the development of the septic tank process in Urbana and Champaign was made at about the same time as that at Exeter, England, the two being entirely independent of each other, though along much the same lines. * * *

"It may be well to say that this system may be used without infringing on any patents. While a company claims to have a patent, the principles of darkness, non-ventilation, submerged outlet, and other features of the septic tank have been in use so long that they certainly are not patentable."

This brings us to the present period, and to the development of the modern septic tank and bacterial contact systems of sewage disposal. Some excellent and very suggestive studies have been made in this field within the last decade, and the actual processes of Nature are probably far better understood to-day than formerly. Many of the professedly new principles and devices used are covered by patents, both at home and abroad; yet a comparison of the actual methods used to-day with those of years ago, regardless of new interpretations or explanations of the forces involved, discredits at least some of the claims of novelty now being advanced.

Enough has been said, however, to indicate that the following processes and operations are old in application in the field of sewage disposal, and hence that their use, *per se*, cannot constitute an infringement of patent rights:

1.—Sedimentation, as shown by the work of Mr. Austin, Dr. Tanner, and many others referred to;

2.—Reduction and liquefaction of the solids, soaps and fats by bacterial or anaerobic action, hydrolysis, fermentation, maceration, etc., by slow passage of the sewage through a tank or chamber, as shown in the different types of tanks and modified cesspools used in Paris and the world over, the Mouras' automatic scavenger, Paglianl's tank, Philbrick's tank, Croes' Lawrenceville plant, Talbot's Urbana and Champaign tanks, the Amherst tank, and many others;

3.—Decantation of the liquid and liquefied portions of the sewage from the tank without disturbing its contents or the surface blanket or sludge deposit in the bottom, by means of the submerged outlets, as shown by the work of Mr. Austin, Dr. Tanner and others, Mouras'

automatic scavenger, Paglianl's tank, Philbrick's tank, Croes' tank, Wheeler's Massachusetts Reformatory tank, and many others cited;

4.—Hydraulic sealing or trapping by submerged inlet and outlet, as shown by numerous examples cited;

5.—Roof or covering, as shown by various examples cited;

6.—Artificial ventilation, as shown by certain of the tanks used in Paris, Wheeler's Massachusetts Reformatory storage chamber, Philbrick's tank, Croes' tank, and others;

7.—Absence of ventilation and exclusion of light and air, as in Mouras' automatic scavenger, Paglianl's tank, and others;

8.– Removal of the sludge without draining the tank, as in Paglianl's tank, the Amherst tank, and others;

9.—Overflow and aeration by weir, as shown in Austin's tank, Croes' Lawrenceville tank, and others;

10.—And many other processes and devices of lesser importance.

To discuss the recent work, however, is beyond the purpose of this paper. The writer has aimed only to present, to those interested in the subject, certain facts, gathered by him in his investigations, which seemed to have an important bearing upon some of the more recent claims for novelty, as well as upon certain old patents now being interpreted in the light of modern scientific development, and in a way never dreamed of by their authors.

DISCUSSION.

Mr. Snow. FRANK HERBERT SNOW, Assoc. M. Am. Soc. C. E.—The author is not the only engineer who has been bothered by claimants to proprietorship in the septic tank. If others have gone into painstaking research of the antecedents of this modern method, it is not known. Mr. Metcalf is the first to submit, in permanent form, for general reference, the results of such investigation, and his expressed hope that this effort may be of service to brother engineers interested in the patent question will undoubtedly be realized.

Modern proprietory methods of sewage disposal are demanding the attention of civil authorities. The septic tank is a modern method, in the sense that it was not recognized as a practical mode of treating a large volume of sewage until about six years ago. It is not a modern method in the sense that the action peculiar to sewage confined in receptacles has been known and observed for many years. The plans on Plate XI show tanks which were constructed for the purpose—as the speaker understands them—of intercepting the solids in the sewage, prior to subsequent treatment of the liquid. It has not been made clear what disposition of the sediment was intended. The advocates of the modern septic tank at first claimed complete liquefaction of the solids, but now modify this claim. That some such action did take place in the old tanks was known. Thus, not all the claims for the modern system are being realized, but, on the other hand, more has been accomplished by the old tanks than was expected or intended by their designers.

The speaker does not think for a moment that proprietary rights in the septic system need be a subject for serious apprehension, but he does consider the patent question a pertinent one, and will confine his remarks to a brief review of the subject as it has come to his knowledge since 1882, when he first became interested in sewage disposal.

The invention of systems of, and appliances for, treating sewage has more than kept pace with the actual development of the science of sewage purification. A great many inventions have been patented. Many of the patents issued are without doubt invalid, many of them are impracticable, and a few may meet a real need which is not universally felt at the present time, because the science has not sufficiently developed.

Mr. Metcalf has referred to patents which cover the subsequent treatment or disposal of the septic fluid, as well as those which refer to the septic tank itself, and the discussion may, therefore, properly take into consideration the aerobic and anaerobic processes.

The speaker has used the septic tank in combination with filters

for several years. Recently, he has had his attention called to the fact that claims for infringement of patent rights were about to be pressed in nine cases in which he is interested in the design and construction of sewage purification works. These claims are so broad that they appear at first to be ludicrous, and their attempted enforcement little less than blackmail; but, nevertheless, the fact of the mere notification of intention to sue is enough to arouse fear in the mind of the alleged infringer, and to place the designer of the plans in an unpleasant position before his client. Therefore, the patent question has forced itself upon some of the members of this Society, already, and it behooves others to consider their premises well before submitting plans for adoption. Mr. Snow.

Sewage left to its natural course will ultimately purify by the agency of chemical changes depending upon bacterial action. The early artificial methods of disposal, like chemical precipitation, were antiseptic in their operation, and inimical to bacterial life. At the present time the whole trend of thought and effort is to discover and meet the conditions which aid and intensify this life. After it was realized that the action could be accelerated by oxygen without the agency of vegetation, the use of sands, over which the sewage could be spread in thin films, thereby increasing the area of oxidation and bacterial action, and in which, by alternately applying sewage to different beds, the necessary supply of oxygen could enter between doses, was investigated, and the results made public property. This work, by the State of Massachusetts, was the foundation of the American system of intermittent filtration.

During this period of evolution in methods, inventions, aiming to cover the use and control of air in filters, were announced from time to time. The speaker has been compelled to believe that some of the alleged discoveries and inventions were not based on knowledge and reasonable experimentations so much as upon theory and desire to monopolize what appeared to be the coming system of sewage purification.

The evil of allowing fake systems to demonstrate their inefficiency at public expense, in preference to taking sound professional advice, has been obviated in Massachusetts by reason of the fact that plans must first be approved by the State Board of Health, whose standards of comparison afford no encouragement, but rather keep the promoter away.

Passing quickly on, experience soon proved that rates of sewage purification by sand filtration depended upon the size of the sand grains, and on the fact that, owing to the fineness of the material, the bacterial action was limited to the upper few inches of the filter; and also that the principal difficulty was found in handling the suspended matters in the sewage. Where fresh sewage was applied, fats and

Mr. Snow. fæcal matter accumulated on the surface of the filter, requiring frequent raking off; where stale sewage was applied, the surface deposits were found to be less and the fatty matters more easily handled.

The great question was the sludge problem: Was it better to separate the sludge from the body of the sewage, and press it, or separate it and apply it to particular beds, or indiscriminately apply it mixed with sewage?

Contemporaneous with these experiences, various patents for handling and disposing of this material were allowed. The most noted contrivance for this purpose is the septic tank. The proprietary right to this system will not be generally conceded in this country, unless sustained in the courts. The reason is, that the process is simply an extension of the action which has long been recognized as naturally taking place in sewage when it becomes stale. The tank is primarily a settling basin. The subsided matters putrefy and give off gases which raise the solids to the surface, where the gases escape and the solids fall to the bottom again, and the rise and fall continue until the solids pass into liquid form through bacterial action. The process is natural. It may be facilitated or hindered. The exclusive right to the natural process may well be questioned. The validity of patents issued for appliances to facilitate the action is not questioned, except in particular cases. Such appliances, placed for general sale, may be included in the engineer's design, and their use safeguarded under terms suggested by ordinary business methods. But it seems that the tank itself, and combinations with machinery or contrivances for automatically governing the same, cannot be used without liability of litigation.

At least one royalty has been paid in this country for the use of the septic process. The engineer who designed the works has sounded an alarm, and a similar payment will be more difficult to obtain hereafter. The case cited affords an example of what an alleged infringer may be willing to do to avoid annoyance, even where the validity of the claim is denied.

It is interesting to consider the claims of the much-discussed "Glover System" of sewage purification. No one has ever been able to tell just what Mr. Amasa Glover, of Brockton, Mass., did invent or discover, and patent. He claimed everything in sight, as the science was developed from year to year. Whether or not Mr. Glover did sufficiently and properly describe his invention, in the specification which was originally filed in the patent office, is immaterial to this discussion. The speaker can testify that Mr. Glover did claim an absolute disappearance of all solids either in the central tank of the system, or the subsequent filters grouped around the tank. He recognized the necessity of oxidation, and provided for a kind of aeration of the filters and also for periods of dosing and rest. Later, when the Massachusetts

State Board of Health began experiments on sands, Mr. Glover seriously considered bringing suit for infringement, until he found out that this could not be done. Subsequently, he decided that the experiments confirmed his claims, and it is a fact that, on the strength of the State Reports, men purchased stock in his company. The patent has long since expired, but there are scores of men living who will testify that Mr. Glover discovered the bacterial system of purification, and, owing to this fact, they were induced to invest money in the process. Mr. Snow.

The speaker does not wish to be understood as confirming in any way this opinion, but he is stating the facts, as he knows them, of this particular claim to the discovery of the septic and bacterial processes.

The date of that patent was 1882, or, perhaps, as early as 1880. In any event, it expired several years ago.

At the present time the best information regarding the preliminary decomposition of the organic compounds in sewage, especially the breaking down of cellulose and fatty matters, is that it must be by anaerobic liquefaction and hydrolysis. The subsequent purification must be aerobic, and, along these lines of oxidizing treatment, progress has been made in the use of filters of coarse material and gates to hold the sewage in contact with the bedding material for stated periods. In this manner the field of bacterial action is extended through the entire depth of the filter, and higher rates are thereby secured. This step has brought out another distinct class of inventions. Patent rights are now claimed on devices for controlling the septic action, and combinations with various kinds of filters, on the filters, sedimentation tanks, and various contrivances and appliances for controlling, distributing, and collecting sewage in its passage through the different stages of the purification works.

Some of these rights are owned by companies organized for the purpose of constructing disposal works. New companies are being organized for this purpose. In every instance patents are the basis for the enterprise. The great field of sewage purification offers inducements to capital, and any plausible scheme to create a monopoly of the business can find financial backing. It is with sewage disposal companies owning patented processes that the engineer will be brought in contact more frequently in the future.

Now, contracts awarded on plans and specifications furnished by each bidder have been held to be illegal, because the law implies a common standard upon which bids shall be compared. As any other method would be contrary to the policy of law and good government, it is usually held that a city must prepare its own plans and specifications, and all patented processes constituting the subject of a monopoly must be excluded from the plans. This law, or this interpretation of the law, if strictly carried out, would prevent many useful appliances and inventions being used on public works, which, in itself, would be

Mr. Snow. poor public policy. The speaker considers a disposal plant to be like a machine, or anything complete in itself, operating to perform a stated amount of work in a stated time and manner, and, when considered in this light, it is a subject for proposal and contract just as much as a pumping engine, steam-boiler, or any other contrivance which can be constructed and set up ready for use by a contractor.

If this be so, sewage disposal companies owning patents will be encouraged to struggle for existence. Proprietary systems will be offered to municipalities, and, in evidence of good faith, promoters will agree to execute satisfactory bonds in almost any amount that the claim for the system shall be obtained.

Such propositions demand consideration, and in time they reach the engineer. Even three years ago he did not have to consider to any extent the utility and validity of patents and proprietary systems, but there is no question now about the necessity of a comprehensive knowledge of these matters.

In conclusion, the speaker does not believe that the threatened suits, referred to at the beginning of these remarks, will ever be brought to trial, but if forced into litigation, he will enter the contest with the assurance of extended co-operation on the part of engineers, and with full confidence as to the final results.

Mr. Hering. RUDOLPH HERING, M. Am. Soc. C. E.—The author has done a commendable service in bringing together this material and putting it in good shape for reference, as it will materially help all those engineers who are interested in this particular subject.

The speaker's impression is that Mr. Philbrick constructed his tanks and sewage disposal plants somewhat earlier than 1883, the date given in the paper. During the eighties, the speaker built two plants, each for the disposal of 20 000 galls. a day, practically on Mr. Philbrick's suggestions. One of these plants was near Philadelphia, the other was for a hotel on an island in Lake George. The experience was very interesting in more than one way. The latter plant was put up early in the spring, before the hotel was opened, and at the end of the season there were several things that surprised him, and for which he did not find an explanation until recently, when the septic tank process became common knowledge.

Mr. Philbrick suggested two settling tanks, into which the sewage first entered, dropping its heavier matter to the bottom and suspending the lighter matter at the surface, this being the ostensible object of these two tanks, which were to be used alternately. The liquid between the two layers of solid matter was, by means of a submerged outlet, passed into another chamber which was the siphon tank. From the latter, whenever the tank was full, the sewage, with almost no suspended matter, was discharged automatically into the pipe delivering it upon the prepared surface of land, where it was absorbed by the

sandy soil. At the end of the summer the speaker was surprised to see so little deposit in one of the tanks, and to learn that the other one had not been used at all, because it had not been necessary. The second one was to be used only when the accumulation of deposits in the first interfered with the natural separation, and required removal. This settling tank was about 20 ft. in diameter, and there was a deposit of about 6 ins. or 1 ft. of sludge and a scum of 2 or 3 ft. It was said that this had been the condition for several months. The speaker could not understand at that time why the scum and deposit increased rapidly at first and then practically remained constant, although the quantity of sewage was greater rather than less. The reason is now known. Mr. Hering.

It seems to the speaker that this tank was practically the same as that which is now called the septic tank. Mr. Philbrick, and also a Mr. Barrett, of Staten Island, had built a number of them.

In 1880 the speaker had the privilege of meeting Mr. Rogers Field, who invented the first practical automatic siphon for sewage discharge. He showed him, at his office in London, the application of this siphon to the disposal of the sewage from country residences, with the same arrangements that Mr. Philbrick published in his little pamphlet.

GEORGE W. RAFTER, M. Am. Soc. C. E. (by letter).—The writer can contribute, to the list cited by Mr. Metcalf, a brief description of a septic tank which has been in use since 1889 or 1890 at the old Colony of Manhattan State Hospital, at Central Islip, N. Y. Its capacity is about 80 000 galls. or sufficient to take nearly the entire day's flow. It is underground, arched over, and thus entirely covered, entrance to it being effected through manholes. The contents are pumped out upon land in about 5 hours. There are no plans, and its exact dimensions are unknown to the writer. The sludge is removed twice each year. Mr. Rafter.

JAMES OWEN, M. Am. Soc. C. E.—The speaker can corroborate Mr. Hering's statements as to the early use of Mr. Philbrick's design for closed tanks, as outlined by the author. About the year 1882, and for some years after, the speaker constructed a number of small sewage disposal plants upon Mr. Philbrick's design, and he was surprised to find out how small was the deposit in the bottom, and how little the accumulation of scum on top, in tanks which had not been cared for. Mr. Philbrick designed these tanks to be systematically cared for, by having the top scum removed periodically. In one particular case, no care had been taken with the plant, and, four years after, it was found on examination that the surface accumulation was very small (about 1 ft.), and that the bottom deposit was less than that, showing that the septic action, which is now recognized, was going on, but was then an unknown factor. Mr. Owen.

Referring to the main point of discussion in the paper, relative to

patent rights, about the years 1885 or 1886, a notice from a lawyer was received by the speaker, stating that he was infringing on Mr. Waring's patent by using the subsoil irrigation system for the disposal of sewage, and it was a great surprise to him, as he had received his early education from an entirely different source, viz., Mr. Philbrick, and as he had never read Mr. Waring's book on the subject, and did not know that Mr. Waring had any patents upon the system. The claim was never pressed, nor was any royalty ever paid.

In regard to the septic tank, the speaker entirely agrees with Mr. Snow, that the knowledge of patents on ordinary engineering structures is a great detriment to the furtherance of such construction by clients who may desire to have them, on account of the dread of lawsuits to which they are liable by the use of a patented article or system, whether the patent is or is not really valid. The septic tank, as originally worked out by Mr. Cameron, was a purely professional design, and he never had a patent on the septic tank principle, but only on the distributing arrangement, which was a purely mechanical device. In this country, however, patents have been granted on the septic principle of the decomposition of sewage, which is really a natural process, guided, of course, by special conditions.

About four years ago the speaker designed and constructed a septic tank having a capacity of about 40 000 galls., and based on Mr. Cameron's idea of the Exeter tank. No plans of the Exeter work were available, and the speaker evolved the construction of his tank from his own ideas of the general requirements for such work. Shortly after the completion of the work a letter was received from a gentleman in Chicago stating that he was interested in the septic tank principle, and desired to see the tank. A favorable answer was sent to this gentleman, the inspection was duly made, and no claim of any patent rights was made at that time. Last year the speaker constructed another septic tank, of about the same capacity, description of which was current in the professional papers, and a notice was sent to him from a Chicago syndicate claiming damages for the infringement of a patent. The claim being repudiated by the speaker, the representative of the syndicate—the same person who originally claimed to be interested in septic tanks—appeared on the scene. After a free discussion of the question, it appeared that the particular patent for septic tanks was issued to the syndicate about six months after the completion of the first tank constructed by the speaker, and the interview closed with a positive intimation on the part of the speaker that he would repudiate any claims for the infringement of the so-called patent. It is very doubtful, in the speaker's mind, whether any patent for septic tanks issued in this country has any validity whatever.

The tank at Champaign, Ill., was constructed before either of the speaker's tanks was commenced. It was practically contemporaneous

with the Exeter tank, and they both emanated from the investigations of the Massachusetts State Board of Health. Mr. Owen.

The speaker is somewhat solicitous about the patent question. In his opinion, it is entirely unprofessional for a civil engineer to get any patent on any work which he has constructed. If an engineer, in exercising his professional function on behalf of his client, should design something novel or new, it does not belong to him, but to his client, and, with a due regard for the proper professional standard, he cannot use it. If he has obtained a patent on work or construction which he has designed outside his professional capacity, he is practically debarred from using it himself, because he has an ownership and an interest in a merchantable article, in the use of which he is debarred by the ethics of his profession.

The speaker wishes to lay down the law very strongly on this patent question, as there is a prevalent idea, especially among the younger members of the profession, that if a professional engineer gets up a new design which has especial merits, it is perfectly proper for him to patent it. The speaker believes that such an act is entirely unprofessional.

Leonard Metcalf, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Snow, in his interesting discussion, has stated that, "It has not been made clear what disposition of the sediment was intended," in the tanks illustrated. The writer had not considered that point as germane to the subject. To clear up any doubts, however, it may be said that the same general methods of disposal were made use of that are to be found to-day. Mr. Metcalf.

Thus, in Henry Austin's plan, the sludge was composted with ashes and used for fertilizing purposes. At the Derbyshire School it was pumped out and distributed on land. The sludge of the Paris tanks was pumped out into tight receptacles, carried to central stations, and from them pumped to large sludge beds in the suburbs, which beds were underdrained like filter beds. The sludge was thus drained underneath and dried out by the air. In most of the other plants, composting, followed by disposal upon land, was the common method of treatment. In the chemical precipitation plants, of course, sludge presses were used.

The writer would confirm Mr. Hering's "impression * * * that Mr. Philbrick constructed his tanks and sewage disposal plants somewhat earlier than 1883," by stating that he has been informed from other sources that such is the case. The writer's only reason for adopting the date 1883 was that the latter was the date upon which Mr. Philbrick published in an engineering magazine a technical article upon the subject, thus giving public expression to his views and making them a matter of "common knowledge."

A few additional examples of early tanks have been brought to the writer's attention by C. C. Brown, M. Am. Soc. C. E.

Mr. Metcalf. The first is a small disposal plant built upon the chemical precipitation system, designed by Mr. Brown for Stamford, N. Y. At the time that this tank was built Mr. Brown was of the opinion that the entire plant was not necessary, and he therefore recommended the construction of the sedimentation tanks only, with the addition of some baffle-boards to prevent a direct surface current. The tank was covered with a plank roof. Mr. Brown says of it:

"This was really accidentally a septic tank; when I visited the town one or two years after it was constructed, the action of the tank was reported to be satisfactory."

The second plant referred to by Mr. Brown was that at Round Lake, N. Y., which was originally run as a chemical plant, but which has also been operated to a considerable extent, as what would now be termed a septic tank. The Soldiers' Home, at Bath, and the plant at Mt. Vernon also make some approach to this system. The Institution for the Blind, at Batavia, N. Y., which has one of the German upward filtration plants, was also modified somewhat, and made to approach more nearly the septic tank arrangement.

In regard to these examples, Mr. Brown says:

"None of these tanks, with the exception of the first two, could be classed as a septic tank, strictly speaking, and none of them was constructed as a septic tank."

Mr. Owen's remarks upon the patent question are most timely ,and are heartily subscribed to by the writer. The latter felt, in presenting his paper, that a broader question than that indicated by the title was really involved, namely, the attitude of the engineering profession upon the question of the patenting of the methods and devices used in any engineering works by their designing engineer.

It is, perhaps, a question of legal rights *versus* moral rights or expediency. No one would question the right of the inventor to government protection and to the legitimate enjoyment and benefit of his discovery; yet, on the other hand, it is often hard to draw the line between true originality or novelty of design and mere adaptation to the problem in hand of the knowledge and discovery of others. This appears to be particularly true in the field of sanitary science, which has seen such marked advance in the past decade. Who, for instance, can say how far reaching have been the results obtained by the Massachusetts State Board of Health in their influence upon the later developments in the bacterial treatment of sewage?

The United States and Germany are the only countries in which a thorough search of records is made before granting patent rights. In Great Britain and her colonial possessions; in France, and in other countries, no assurance is implied in the granting of patent rights that the patent has not been anticipated. In the United States, however, an examination is made, not only of the specifications of the United

States, but also of those of foreign countries, and the scrutiny is extended to general scientific literature. Mr. Metcalf.

Therefore the granting of patents in this country gives to claims an authority—bearing in mind the limitations of the patent office and the fact that existing plants frequently fail of notice in engineering journals—which the premises sometimes do not warrant. Yet this authority is often sufficient to prevent the adoption of proposed engineering plans by municipalities on account of fear of subsequent, or sometimes threatened, litigation.

From the point of view, therefore, of professional interests, not less than of the advancement of science, it would be most unfortunate were American engineers to outgrow their old objection, or perhaps prejudice, to the patenting of new or improved engineering methods and devices, and to adopt the English system of wholesale patenting. Mr. Owen has presented the case, from the standpoint of professional ethics, in a clear and forcible manner.

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No. 910.

DESCRIPTION OF COOS BAY, OREGON, AND THE IMPROVEMENT OF ITS ENTRANCE BY THE GOVERNMENT.

By William W. Harts, M. Am. Soc. C. E.

Presented May 15th, 1901.

WITH DISCUSSION.

The recent completion of the north jetty at Coos Bay, Ore., and the satisfactory results following its construction make it appear of value to give some description of this work and the conditions of the entrance prior to the commencement of operations, especially as a study of certain methods of construction, satisfactorily used here and in neighboring localities, may be found of some value in designing works at other similar places.

Coast.—The Pacific Coast, north of San Francisco, embracing more than 800 miles, is not abundantly supplied by nature with suitable harbors. The Puget Sound ports and the mouth of the Columbia River on the north and San Francisco Bay on the south are the principal harbors available for deep-draft and ocean-going ships, and the entire coast line has few other harbors which are susceptible of improvement, at a reasonable cost, to the extent of making them suitable for more than moderate-draft vessels.

This part of the coast is remarkable for its boldness and continuity. From Port Orford, near the California State line, north to the Columbia River, the coast is a continuous, almost unbroken, line, but, even so, consists of a succession of bold headlands and sandy reaches, broken by but few bays or rivers. The bights between the headlands are filled out with bare sand reaches, usually without vegetable life of any kind, and frequently rising in dunes of considerable height. The action of the elements in wearing away the headlands seems to be also directed toward filling out the coast lying between them, so that a nearly continuous shore line is maintained.

The Coast Range of mountains lies close to the line of the coast, and the headlands appear to be portions of these mountains not equally worn away by the agencies which have eroded the remainder. These mountains are mostly soft, metamorphic sandstone of the tertiary age, and are believed to overlie granite, from the outcrop at Point Reyes, In passing up the coast, basalt is encountered at Cape Foulweather, presumably overlying this sandstone formation. These sandstone headlands have supplied, by their disintegration and by the erosion by wind and sea, an abundance of fine, loose sand which is found in great quantities everywhere along the coast from Cape Blanco to the Columbia River, and even to Puget Sound. The action of the sea waves and tidal currents upon the sand of the ocean bottom near shore and the action of the wind above the water line is believed to have caused to a great degree the singular character of the bays and river entrances and the existence of the usual semi-circular or crescent-shaped bars across their openings into the ocean.

The severe action on the shore by the ocean waves during storms is increased by the sharp slope seaward of the ocean bottom and by the lack of other land for thousands of miles to the westward and southward to break the force of the seas. From the Columbia River to beyond Coos Bay the contour of 500 fathoms depth in the ocean is only about 30 miles off shore. Near Coos Bay the 50-fathom contour is only about 5 miles off shore, and the 1 700-fathom contour is only about 50 miles off shore.* Compared with many other coasts this slope seaward is very abrupt, permitting great depths to be found comparatively close in. This deep water near shore and the tremendous reach of the ocean to the westward and southward, whence come

* U. S. Coast, and Geological Surveys.

the worst storms of this region, are features that help to make this coast one of the most exposed in the world. The full force and violence of the waves is therefore directed against or along this shore, uninterrupted by shoal water, and transmitted probably for hundreds of miles in the open sea.

Thomas Stevenson, the English harbor engineer, has even expressed in a formula the relation between the height and force of waves and the distance of exposure in miles ($h = 1.5 \sqrt{d}$).

Storms.—It was thought for a long time that no harbor works, along this part of the Pacific Coast, extending into the open sea, unless unduly expensive, could withstand the force of these waves, and for this reason improvements at certain harbors were formerly attempted by constructing deflecting jetties inside the entrances, where they were more or less protected, in the hope that this work would better the channels across the bars without extending the construction beyond the general line of the shore. These works, unfortunately, were only partially successful.

The worst storms along this part of the Pacific Coast seldom make a lee shore, but come from the south and southwest, thus enfilading the coast. They usually start in the south or southeast and shift to the westward until they reach a quarter almost due southwest. In the winter the storms are occasionally very severe and often of long duration, and at this season almost invariably blow hardest and longest from the southwest. In the summer, on the other hand, the wind is generally northwest, and the summer gales come usually from that quarter. These storms and winds often set up a very noticeable off-shore current, said to amount at times to 2 knots, which, accordingly, is northward in the winter and southward in the summer, being more marked and of longer duration to the northward. At certain localities the conformation of the shore line induces eddy currents, opposite in direction and closer in shore.

As to the exposure of this portion of the Pacific Coast, the testimony of experienced navigators, in 1881, before a Board of Engineer Officers convened to report on a site for a harbor of refuge on the Pacific Coast, went to indicate that there were few localities in the world where the seas were worse during storms. On the California Coast these dangerous storms are less violent, less frequent and shorter than on the Oregon and Washington Coasts, but north of California,

especially from Cape Blanco to Cape Flattery, they are frequently very violent and sometimes succeed each other in rapid succession for weeks. Some navigators give Cape Foulweather, about 100 miles south of the Columbia River and 100 miles north of Coos Bay, as the locus of the worst storms. Captain Maury, Commander of the *City of Tokio*, stated to this Board:

"There is no such sea in the world except south of the two capes or in the Indian Ocean. I have seen the sea break in 8 to 10 fathoms. There is no comparison between the heaviness of the southwest sea on this (Pacific) coast and that which has damaged the works at Cherbourg and other places."

Captain Simpson stated:

"There may be such swells in other oceans, but I have never seen them."

Several other vessel captains testified that they had seen the waves on this coast break in 8 to 10 fathoms, and some said they had seen them break in 15 fathoms. Sometimes a heavy swell breaks on the shore in a dead calm, probably the result of a distant storm, the propagation of the waves of which is favored by the great depth of the sea.*

Harbors.—The character of the harbors on the Oregon and Washington Coasts is noticeably similar. In almost every case the entrance from the sea is between a high, bold headland on the one side and a long, low-lying tongue of sand on the other. Sometimes this tongue lies to the northward of the opening, as at Coos Bay, and sometimes to the southward, as at Tillamook and Yaquina Bays and the mouth of the Columbia River. In all cases at some distance out from the opening is to be found a crescent-shaped bar, more or less continuous, and usually completely encircling the entrance. These bars, from their insufficient depths, form the principal obstructions to these harbors, and on account, also, of the shoal water on these bars, the waves during storms often break entirely across the entrances, making the crossing in or out by vessels very hazardous, even when depths are sufficient; or, if the seas are not actually breaking, the water becomes so rough that crossing is unsafe. In such cases ships are "bar bound," with ample depths on the bar. The improvement of these harbors, therefore, usually contemplates the formation of a deep channel through the bar in some selected place.

* See Annual Report, Chief of Engineers, 1881.

Bar Formation.—The main causes of the formation of these bars seem plain. The ocean bottom along the shore for some distance seaward is composed of fine sand which is easily moved. The waves of the westerly and southwesterly storms, in breaking off-shore, dislodge great quantities of this sand and carry it shoreward to be piled on the beaches or deposited by the waves and littoral currents in any exposed openings. Oftentimes small creeks are completely closed in this way. In these cases the damming effect of this sand thus piled up goes on until the quantity and difference of head of water inside the bar and outside is sufficient to cut a new channel.

In the mouths of large bays the action is similar, the shoaling continuing most actively during the storms; erosion, however, to a greater or less extent, taking place afterward by the scour, due to the water passing in and out of the bay on the tides. Unquestionably, much sand is also carried into the bays on the flood tides, and large quantities of this sand are then deposited in sheltered places within the bay; but large quantities are also carried back into the ocean on the ebb, the well-known tendency of the channels in tidal estuaries and bays, to maintain their depths or even deepen them, being usually sufficient to prevent any deposit in the path of the currents themselves.

The channels, in reaching the ocean, are also sometimes crowded far to the northward, as at Tillamook Bay, by the encroachment of the moving sand, due to the southwesterly storms and shore currents, until the distance the water is compelled to traverse in getting out of the bay on an ebb tide is enough to create a hydraulic head nearer the old entrance sufficient to cause a new exit to be cut out. In some cases, as at the mouth of the Nehalem River, the channel is crowded to the southward in a similar manner by the building up of the spit on the north side of the entrance. The direction taken by the free end of these sand spits, as a general thing, indicates the direction of the resultant sand movement at that locality.

This bar-forming action during storms has been observed at Coos Bay, where the action of the waves in stirring up the sand of the sea bottom was plainly visible in the discolored waters off-shore, and where, on one occasion, during a severe storm, the bar shoaled 4 ft. in a single flood tide and scoured out again on the next ebb. Again, at Coos Bay, during a long-continued period of southwesterly storms, in November, 1899, the channel over the bar shoaled to 9 ft., but on the cessation of

the storms during the next month the scouring effect of the tides soon removed this shoal down to the former depth. At the Siuslaw River, the entrance is continually being forced northward, in all for a distance of over a mile, by the constant movement of the sand. This action continues during, perhaps, several seasons, until the channel finally breaks through the bar thus formed, opening a new exit farther to the southward, which again passes through a similar cycle.

It thus appears that at all these harbors there are two main forces constantly at work, one, the effect of the waves, usually in westerly storms, and of the local littoral currents, to fill up or shoal the entrances to the rivers and bays, and the other, the scouring effect of the tides passing in and out, tending to keep the entrances open. If there were no tidal action, the openings into the bays would probably close more or less completely, depending on the amount of fresh water passing out, and if there were no shoaling action of the sea and shore currents, the tidal scour would be likely to maintain entrances of ample depths. The extent of this tidal action, of course, is limited, and its location and amount are governed very largely by such features as the extent of the tidal prism within the bay, the shape of the harbor throat or entrance and the existence of littoral currents in the ocean outside. On an ebb tide the volume of water passing out through the harbor throat, where the width is naturally contracted, has a bottom velocity usually enough to scour and maintain considerable depths. At Coos Bay these depths are in places as much as 60 ft. in the harbor throat, and the surface velocities observed are often as much as 4½ miles per hour. When this tidal stream, however, is free to change direction and diverge on emerging into the ocean the consequent changes of direction, the meeting with outside currents, or the absorption of the energy of the moving water by the ocean, reduces the velocity and widens its area of application over the entire crest of the bar, so that the scouring and transporting effect of the water is thus dissipated. When the velocities of the ebb tide are reduced in the ocean so as no longer to pick up and move outward the particles of sand composing the bottom, the bar is no longer eroded. This, in a certain measure, explains the usual crescentic shape of most of these bars.

It may, therefore, be broadly stated that the bars lying in front of the Pacific Coast harbors are apparently mainly caused by the result-

ant action of the sea on the one hand and that of the tides on the other, while, in all cases, the deposit of silt or other matter in suspension brought down by fresh-water currents, as at the mouth of the Mississippi, is a means of building up a bar that appears to be of no special importance in this region.

Improvement.—This diagnosis of the difficulty suggests the remedy. As the forces going to build up the bars are intermittent, but constantly recurring, no temporary expedient, such as dredging, would be of any special value, owing to the extreme likelihood of such work being completely nullified by the effect of a single severe storm. Any improvement, to be valuable, must, therefore, affect the causes which make the bar, and control the other natural forces constantly at work, so as to maintain the desired depths. If works can be devised to prevent the encroachment of the sand on the channel and at the same time prevent the divergence and dissipation of the effect of the ebb tides, by confining and directing these tidal currents at a selected point of the bar, improvement may be confidently expected.

The method of accomplishing this by jetties was first commenced on the Pacific Coast in 1881, at Yaquina Bay, lying on the Oregon Coast, about 88 miles north of Coos Bay. Here, the entrance to the bay in its original condition was obstructed by the usual enveloping bar on which the best depths, the greater part of the time, were only 7 ft. at mean lower low water. The best depths were found usually in a north channel and in a south channel, both sharply skirting the shore in opposite directions before turning seaward. Occasionally, depths even better than this were found, but they invariably relapsed into their usual condition.

The construction of a single south jetty was determined on first, to close the south channel and direct this volume of water over the bar at such a point as to produce a new entrance along a line nearly due west. This location was chosen partly with a view to make it easier for vessels in passing in and out. The construction of the south jetty seemed sufficient to shut off the supply of moving sand, borne northward along the coast by the currents and by racing waves in storms, but did not give the requisite amount of contraction and direction to the ebb tide, as much water escaped through the old north channel which it was later found necessary to close before much improvement on the bar resulted.

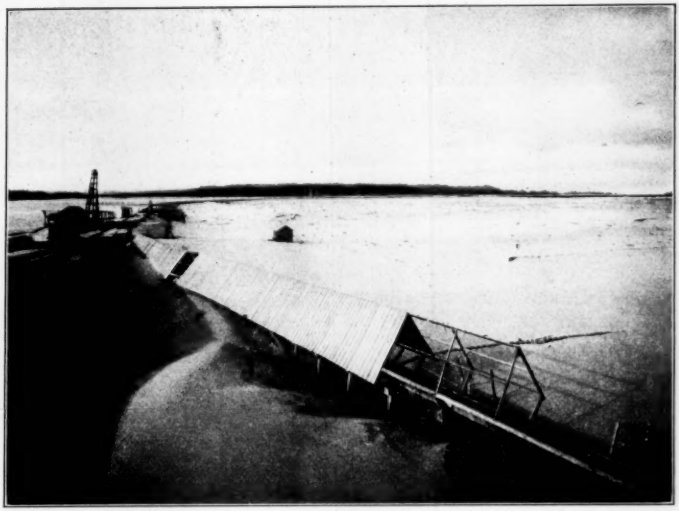


FIG. 1.—GENERAL VIEW OF JETTY, COOS BAY, ORE.

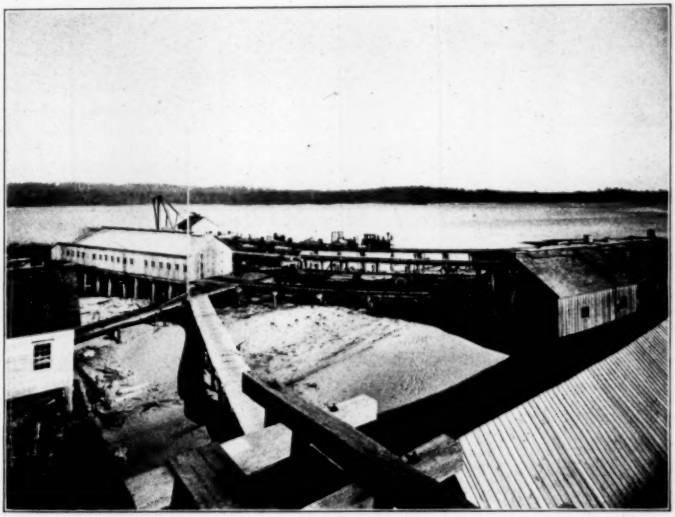


FIG. 2.—GENERAL VIEW OF LOADING WHARF, COOS BAY, ORE.

It was stated before commencing work that the improved depth that could be expected would be at least 10 ft. at mean lower low water, or 17 ft. at mean high tide. After the completion of the south jetty, but long before the completion of the north jetty in 1896, a channel straight out to sea having a least depth of 14 to 15 ft. at mean lower low water, or 21 to 22 ft. at high tide (the mean tidal rise above mean lower low water at this harbor being 7.1 ft.), was produced by the jetties, which depths have been maintained ever since, with the exception of a short period in the spring of 1899, when temporary shoaling of about 2 ft. was reported.

These jetties were the pioneers of the Pacific Coast harbor works. Here, for the first time on this coast, rip-rap jetties were extended out into the open sea. Even in the manner of construction, the jetties at Yaquina Bay developed the type found so useful elsewhere on this coast. At first, it was attempted to build the rock jetties from scows and lighters, as is done on places on the Atlantic and Gulf Coasts. The effect of the heavy seas encountered was too great. Likewise, the plan of using cribs was doomed to failure. The plan finally adopted, first used here in the United States, was the invention of J. S. Polhemus, M. Am. Soc. C. E., Assistant Engineer, in local charge of the work. It consisted in building a tramway from shore, 20 or more feet above the water, carrying double, narrow-gauge tracks, on which the stone for the enrockment was carried in small cars hauled by a locomotive. This tramway required the use of a pile-driver running on the tramway, overhanging far enough to drive the piles of the next bent beyond the existing end of the tramway.

At Yaquina Bay, also, the plan of building brush mattresses while they were suspended under the tramway, also the invention of Mr. Polhemus, was first adopted in the United States. The general methods of doing work which were originated here, although improved in many minor ways afterward, have never been far departed from in similar works on this coast. This work at Yaquina Bay has been eminently successful, from an engineering point of view.

The use of two jetties converging seaward, found so satisfactory at Yaquina Bay, has been influential in causing to be included, in the projects for the improvement of most of the Pacific Coast harbors now under this kind of improvement by the Government, the provision for a second jetty, should a single one prove insufficient. At Humbolt

Bay, Cal., Yaquina Bay and Coquille River, Ore., two jetties have already been found necessary, while at Coos Bay and Columbia River, Ore., and Grays Harbor, Wash., the natural conditions are such that one jetty has already accomplished so much improvement that the necessity for the second jetty is not yet apparent.

There are good grounds for believing that a single jetty performs only a partial service, but at the Columbia River the rocky point at Cape Disappointment acts as a second natural jetty, and at Coos Bay this partial service performed by one jetty has been sufficient thus far, while at Grays Harbor the same favorable results are promised by a partial control of the ebb currents. The trace of the jetties, which side of the entrance to build from first, the length to which they should be extended, the distance apart at their outer ends, are all questions upon which local considerations have so much bearing that it does not seem possible to formulate any fixed rule for them. The questions for an engineering solution must also be affected by consideration for the needs of navigation; for the easy use of the improved channels by vessels is of prime importance.

Objections.—The supposed need of extending the jetties seaward from time to time, to meet the advancement of the foreshore caused by the jetties, is given by some as an objection to this plan of improvement. Such jetty extension has been found necessary already at the mouth of the Columbia River to maintain the improved depths, but nowhere else thus far on this coast. The sand collecting in the groin or angle between the jetty and shore causes at first, in nearly every case, a rapid advancement seaward of both the high- and low-water lines. This sand also backs up the jetty, usually rendering a very light rubble wall ample in strength for all purposes. The objection is made that on account of this action the jetties only transfer the original trouble to another place by pushing the shore lines and the bar farther seaward, as soon as the collected sand has filled the groins. This objection is not very well sustained by the evidence of the facts. While it may not be possible in every case to determine beforehand exactly the length of jetties necessary for the permanent improvement of a special locality, it appears only reasonable to believe that there is a length for each harbor on this coast beyond which it will not be necessary to extend them, the effect of the littoral currents

FIG. 1.—NEAR VIEW OF ENROCKMENT, COOS BAY JETTY.

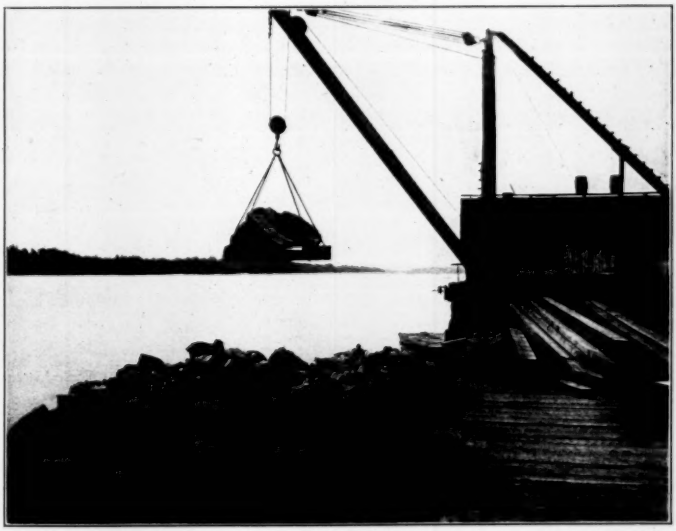


FIG. 2.—UNLOADING STONE FROM SCOWS, COOS BAY JETTY.

and wave action at their ends being sufficient to keep the entrances clear.

It is undeniable that the construction of jetties at the mouth of a bay produces a violent change in the natural condition of the entrance, and that it requires some time for the natural forces to find a new equilibrium; but at Yaquina Bay, the oldest improvement on this coast, the advancement of the shore line has stopped, and, for more than seven years, depths considerably in excess of those stated for the project have been maintained. There is no evidence that any extension of the jetties at this place will be needed to continue this favorable condition indefinitely. It is believed that at other similarly improved harbors, as soon as the proper length of jetties is secured, a similar stable condition will eventually obtain.

Other engineers have feared that two jetties interfered with the easy influx of the flood tide, and might damage the interior of the bay by lowering the high-tide level and lessening the scour. This result has not been found in practice on this coast. On account of the improved channel through the bar, caused by the jetties, or for other reasons not understood fully, the mean tidal range has been noticeably increased at Coos Bay and at the Coquille River, while at Yaquina Bay no reduction in tidal range is noticeable. It is of course conceivable that in laying out the plan for jetties the distance apart at their outer ends might be made too small, but with double jetties properly designed it appears that no further effect is produced than would be caused by moving the throat or the gorge to a new point.

Description of Coos Bay.—Coos Bay lies on the Pacific Coast of Oregon, about 200 miles south of the mouth of the Columbia River, and about 440 miles north of San Francisco. It is the most important harbor between San Francisco and the Columbia River, excepting, perhaps, Humbolt Bay, in Northern California, the commercial value of which is somewhat greater. Coos Bay derives its commercial importance principally from the beds of coal and extensive timber lands found in close proximity to the bay, which furnish the chief exports. The coal is not of the best quality, being a lignite, but in the great scarcity of coal on this coast these measures are valuable. Large forests of Oregon fir, much white cedar and considerable myrtle are found in the vicinity.

The commerce of the bay during the calendar year 1899 was as shown in Table No. 1.

TABLE No. 1.—RECEIPTS AND SHIPMENTS, COOS BAY, ORE., FOR 1899.

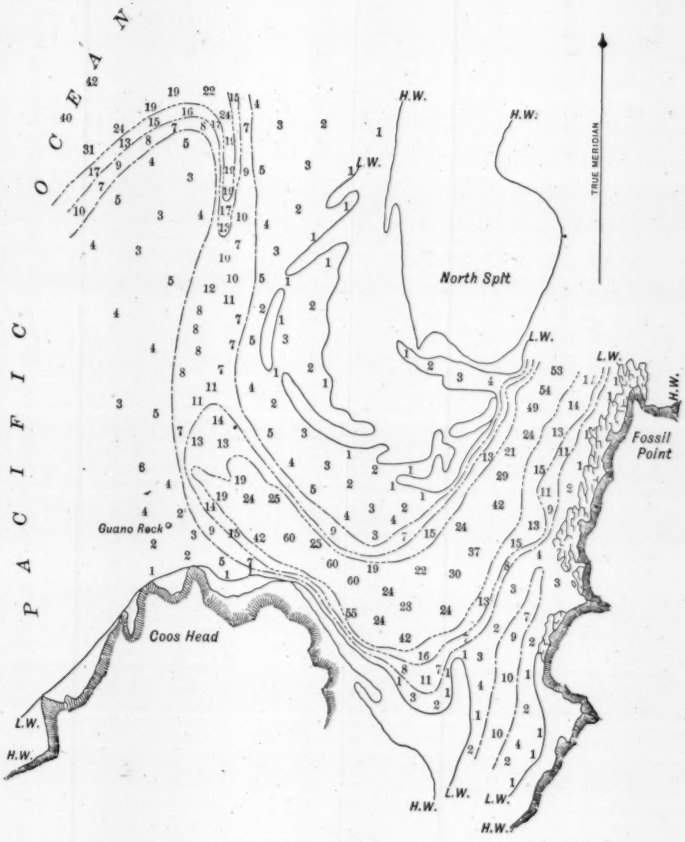
ARTICLES.	TONS.	ARTICLES.	TONS.
RECEIPTS.			
Miscellaneous freight	10 460	Shipments—(*Continued*).	100 396
		Apples and potatoes	501
SHIPMENTS.		Butter and cheese	554
Coal	56 717	Hides and leather	84
Lumber (27 082 815 ft. B. M.)	40 625	Cattle and hogs	269
Laths	282	Fish	485
Firewood	50	Woolen goods	56
Matchwood	164	Miscellaneous	3 762
Piles, poles and spars	2 553		
Cranberries	5	Total shipments	106 107
	100 396		

The value of this commerce is estimated at $2 558 000.

The only connection between the area tributary to Coos Bay and the outside world is by water, there being nothing thus far but wagon roads (almost impassable in winter) over the Coast Mountains connecting with the rail lines of the Southern Pacific Railroad between Portland, Ore., and San Francisco, Cal. Practically all the commerce of this region, therefore, passes over the Coos Bay bar.

Surveys of Coos Bay, made in 1863, 1879, 1885, 1894 and 1899, are shown in Figs. 1 to 5.

The tidal area of the bay is 24 sq. miles. There are no rivers of any special importance emptying into it. The mean range of the tide at the bar is 4.8 ft. The average mean discharge on an average ebb tide is estimated to be about 133 000 cu. ft. per second. The discharge on neap tide is 96 000 cu. ft. per second, while that on maximum tides is about 310 000 cu. ft. per second; ranges of 8 ft. above zero and 2 ft. below zero being occasionally observed in a single day. Surface velocities of 4½ miles per hour on low ebb tides have been observed. The entrance to the bay from the ocean lies between a high, rocky headland on the south called Coos Head, and a long, low sand spit on the north; the width of the opening at the natural throat or gorge being about 1 900 ft. Coos Head is a bold, timbered headland about 60 to 80 ft. above low water, and is composed of a soft, argillaceous sandstone. It extends to the southward for a couple of miles, terminating at Cape Arago, where the Government



ENTRANCE TO

COOS BAY, OREGON

U.S. Coast and Geodetic Survey of 1863.

Soundings are in feet below mean lower low water.

Mean rise of tide above plane of reference, 5.6 feet.

SCALE

1000 500 0 1000 2000 3000 4000 Ft.

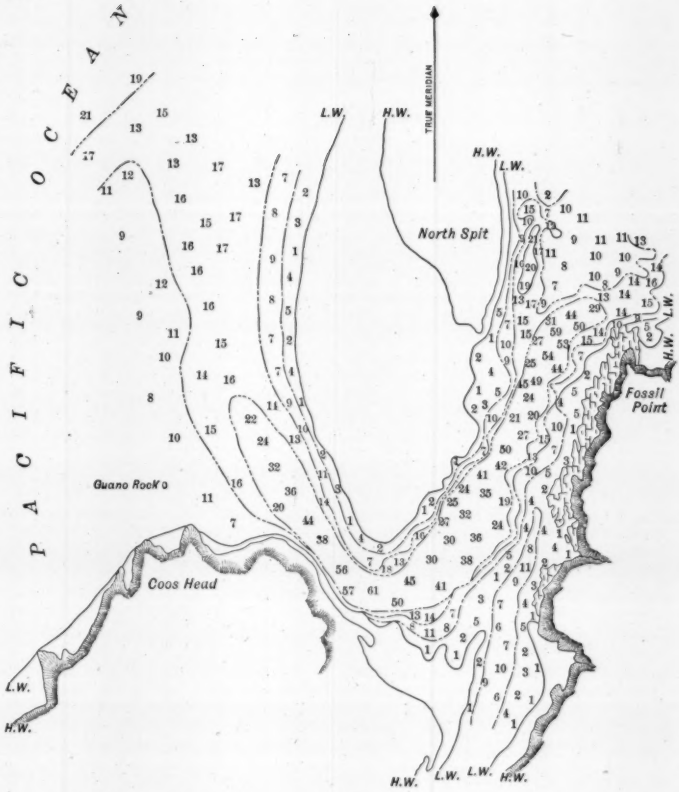
FIG. 1.

maintains a lighthouse. The sand spit to the north of the entrance is about 6 miles long and from ½ mile to 1½ miles wide, with hillocks or dunes of from 20 to 30 ft. in height. It separates the bay from the ocean. The sand beach, of which this spit is the southern end, extends unbroken to the Umpqua River, a distance of 20 miles to the north.

In its original condition, the sand bar in the ocean, obstructing the entrance to the bay, carried depths of usually not over 9 ft. at low water, as shown in the surveys of 1863 and 1885. Occasionally, better depths would prevail, when new and more direct channels through the bar were scoured out, but they were temporary only. The channel was likewise as subject to variation in location as in depth. The encroachment of the sand of the north spit oftentimes forced the portion of the channel just inside the entrance down into the opening into South Slough (see survey of 1879, Fig. 2), and the currents thus impinging against Coos Head were turned sharply to the northward.

This tendency of the north spit and that of the south spit to extend themselves across the entrances were two influences which helped to keep the channel in its northern location. This action continued until the channel, after emerging from the entrance, frequently had a location following close to the west side of the north spit for over a mile before turning to sea. This location usually gave insufficient depths, on account of the great waste of the ebb over the long bar, in addition to forcing vessels to pass "broadside on" to the heavy seas for over a mile before being able to cross out to sea. The survey of 1879 shows this channel crossing what is now the line of the present jetty, nearly a mile inside of its present sea end. Occasionally, in severe storms, this north channel would be closed at its outer end, and a new channel would then break through the bar on a line nearly west from Guano Rock. In this location best depths were always found; occasionally, as much as 30 ft. at high water being reported. This is practically the line of the present improved channel. This new location and depth would last usually but a short time, the channel then working again to the northward in its cyclic change of position.

In addition to the sand carried into the bay by storms and currents, great quantities were blown in during the summer by northwest winds. This sand, coming originally from the ocean



ENTRANCE TO
COOS BAY, OREGON

Survey of Engineer Department, U.S. Army, August, 1879.

Soundings are in feet below mean lower low water.

Mean rise of tide above plane of reference, 5.6 feet.

SCALE

1000 500 0 1000 2000 3000 4000 Ft.

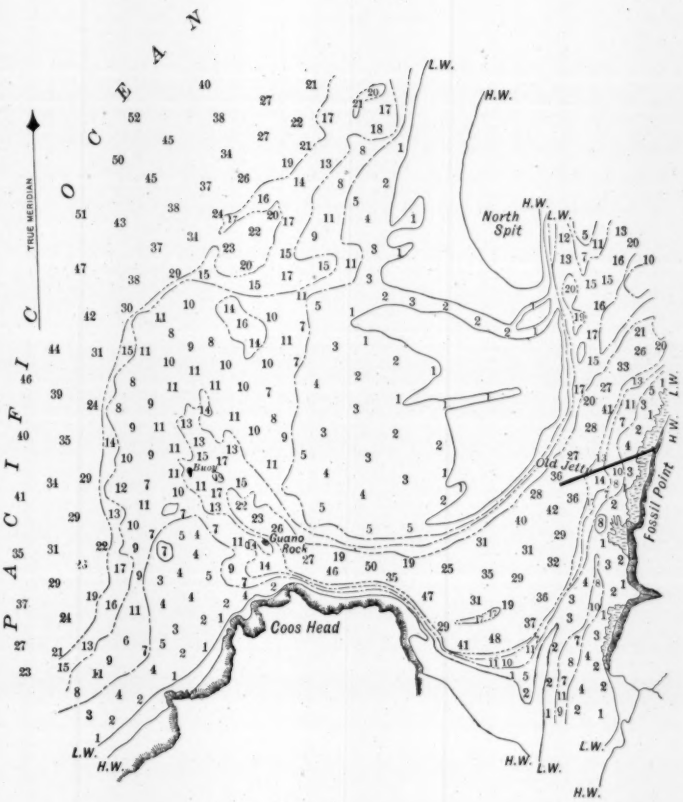
FIG. 2.

bottom, and having been thrown up on the beach in storms, after drying in the air, is easily picked up by the wind, and great quantities are blown into the bay. Some of it is deposited in the bay, and much of it is carried by the tidal currents out to sea where it is deposited on the bar or again thrown on the beach, to again pass through this cycle. The checking of this action was an additional problem.

Plans of Improvement.—The earliest plan of improvement, adopted in 1879, was by the construction of a jetty inside the entrance, where it was protected from storms, and extending nearly westward from Fossil Point (see 1885 map, Fig. 3). This jetty was built of rip-rap rock to a length of over 1 700 ft., and caused a cutting away of the end of the north spit, but had no useful effect on either the direction or depth of the bar channel. This project was abandoned in 1889.

The present plan of improvement of Coos Bay was devised in 1889 by a Board of Officers of the Corps of Engineers, U. S. Army, consisting of Colonel G. H. Mendell, Major W. H. Jones and Major Thomas H. Handbury. This project provided for the construction of two high-tide, rubble-stone jetties, laid as far as practicable on brush mattresses, one extending out from Coos Head, a distance of 4,200 ft., and one from the southernmost end of the north spit, 9 600 ft. long, both converging toward the bar and terminating at a distance apart at their outer ends of 1 500 ft. The function of these jetties was to control the sand movement at the entrance to the bay, to direct the tidal scour upon a special part of the bar, with the expectation of securing a minimum depth of 20 ft. over the bar at mean lower low tide, and afford an entrance along the most navigable lines. The project also included experiments looking to the holding of the sands of the north spit by means of suitable grasses, trees or sand fences, as seemed best, with a view to reclaiming this sand spit and checking the drifting sand. The total cost was estimated at $2 466 412. Between September 19th, 1890, and March 3d, 1899, inclusive, five appropriations, amounting to $675 000, were made. This amount has now completed the north jetty as planned at less than half the estimated cost.

After a careful study of the local conditions, it was considered best to build the north jetty first, leaving the south jetty to be constructed when the needs of the harbor made it necessary. In this way, the large



ENTRANCE TO

COOS BAY, OREGON

Survey of Engineer Department, U.S. Army,

July, 1885.

Soundings are in feet below mean lower low water.

Mean rise of tide above plane of reference, 5.6 feet.

SCALE

1000 500 0 1000 2000 3000 4000 5000 Ft.

FIG. 3.

and more important quantity of moving sand could be checked and impounded, as well as the tendency of the channel to turn to the north stopped. It was clearly evident that the north jetty would certainly be necessary, but, before commencing work, it was not so plain that the south jetty could not be dispensed with.

The line of the north jetty follows the general line of the shore end of the north spit, in a curve of large radius, until it reaches a point 1 900 ft. from Coos Head, from which it runs out to sea in a westerly direction along a straight line. The total length of the jetty is 9 600 ft., the outer end being stopped on what was originally the 20-ft. contour before construction commenced, or just short of the bar proper, it being assumed that the energy of the current, conserved and directed by the jetty, would be effective at a considerable distance beyond the jetty's end. On quiet days at low ebb, the current, running straight to sea, is now plainly traceable for a long distance beyond the end of the jetty. A general view of the jetty is shown in Fig. 1, Plate XII.*

Construction.—The jetty construction consisted in dropping along the line of the jetty large rocks loaded on cars from scows and drawn out by a locomotive along the track on the overhead tramway. The tramway was built on pile bents, each consisting of four heavy piles driven 20 ft. or more into the sand bottom, and capped with a 12 x 12-in. x 22-ft. fir stick and sway-braced by 3 x 9-in. pieces of fir. Along these caps were placed four stringers, 12 x 16 ins. x 32 ft., breaking joints, and on these were laid 30-lb. steel rails for the tracks. Originally, no ties were used, but later, when a greater speed for the trains was desirable, 3 x 6-in. ties were put in. These two tracks were each of 3-ft. gauge, and were spaced 13 ft. apart, center to center. The track started with an elevation of 22 ft. above low water at the receiving wharf, but rose on a slight up-grade until a height of 28 ft. above low water was reached at the outer end. This was the least height at which work could be carried on without much interruption from the waves.

All piles were sunk into place by means of a hydraulic jet operating from a revolving pile-driver† running on the four rails, and having enough overhang to drive a new bent 16 ft. in advance of the end of

* See, also, paper by T. W. Symons, M. Am. Soc. C. E., entitled "Jetty Harbors of the Pacific Coast," *Transactions*, Am. Soc. C. E., Vol. xxviii, Plate xxiv.

† A view of this pile-driver may be found in Mr. Symons' paper, "Jetty Harbors of the Pacific Coast," in *Transactions*, Am. Soc. C. E., Vol. xxviii, Plate xxii.

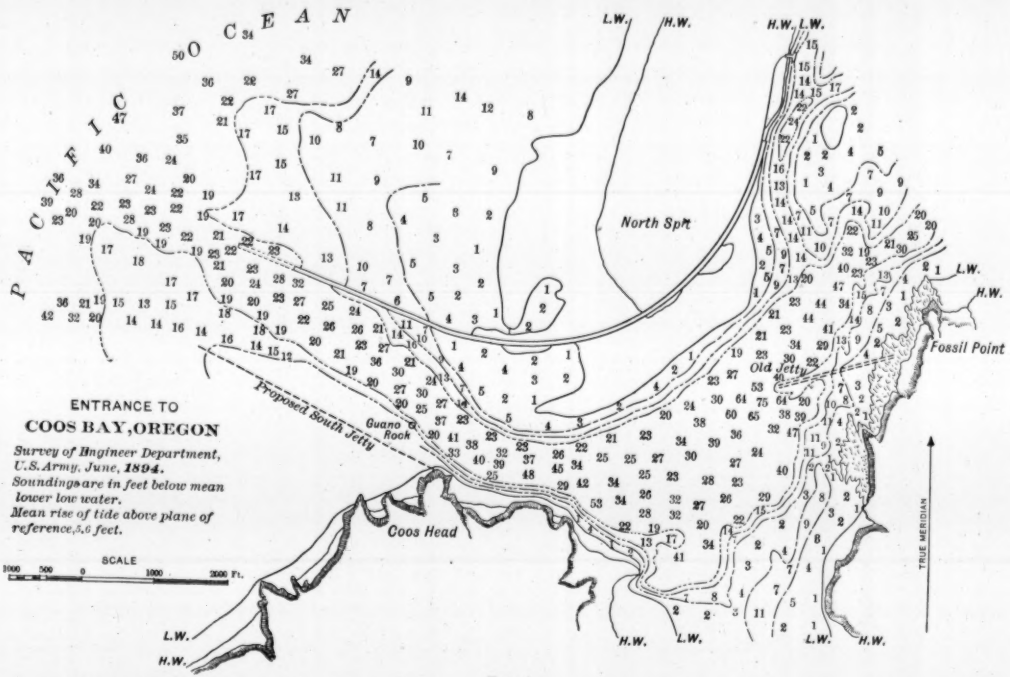

PACIFIC OCEAN
ENTRANCE TO
COOS BAY, OREGON
Survey of Engineer Department, U.S. Army, June, 1894.
Soundings are in feet below mean lower low water.
Mean rise of tide above plane of reference, 5.6 feet.
SCALE
1000 500 0 1000 2000 Ft.
Proposed South Jetty
Guano Rock
Coos Head
North Spit
Old Jetty
Fossil Point
L.W.
H.W.
TRUE MERIDIAN

FIG. 4.

the rails. The piles were 18 to 20 ins. in diameter at the butt, and varied in length from 45 ft. at the shore end of the jetty to 80 ft. at the outer end of the tramway. They were sunk to grade, so that no cut-off was necessary, thus considerably expediting the work. As many as five bents were placed in a single day, under favorable conditions, and 1 260 ft. of tramway, complete, were constructed in one month.

This tramway construction caused a scour around the piles as fast as built, so it was followed as soon as practicable with brush mattresses to hold the bottom until the rock could be dumped. These mattresses consisted of brush fascines, or round bundles, 20 ft. long and about 1 ft. in diameter. They were bound together into mattresses between layers or grillages of poles after being brought to the needed thickness by laying several layers of fascines at right angles. At each pole crossing, the upper and lower pole grillages were bound tightly together with stout rope lashings passing through the body of the fascines, in this way compacting the mattress. The mattresses to be placed between the outside rows of piles were made below the tracks, the lower layer of poles upon which the mattress was built being suspended for this purpose from the tramway by ropes. Upon completion, the mattress was lowered to place, weighted with small rock and sunk. The side mats were made on shore, brought out on a car, dumped overboard, pulled into place by lines, weighted with small rock and sunk in place. The side mattresses were usually about 20 ft. square, and the center ones were from 48 to 80 ft. long by 20 ft. wide. They were usually made from 3 to 5 ft. thick. The main enrockment then followed.

The rock was in pieces weighing on an average about 2 tons*, and was a sandstone of only moderate hardness, weighing about 145 lbs. per cubic foot. It was quarried on Coos River, about 23 miles from the jetty.

Seven scows, costing about $2 000 each, and each carrying about 290 tons, were built for this work by the Government. These were loaded and towed to the jetty, so that two scows each day could be unloaded. For several months, deliveries of 16 000 tons per month on the jetty were made, under favorable conditions. Up to 1896 the Government contracted for supplying the rock on scows at the receiving wharf on the north spit and unloaded it by hired labor, but, after this date, contracts were made for the stone in place on the jetty.

* A view of the dump cars, loaded with rock, may be found in Mr. Symons' paper, "Jetty Harbors of the Pacific Coast," in *Transactions*, Am. Soc. C. E., Vol. xxviii, Plate xxiii.

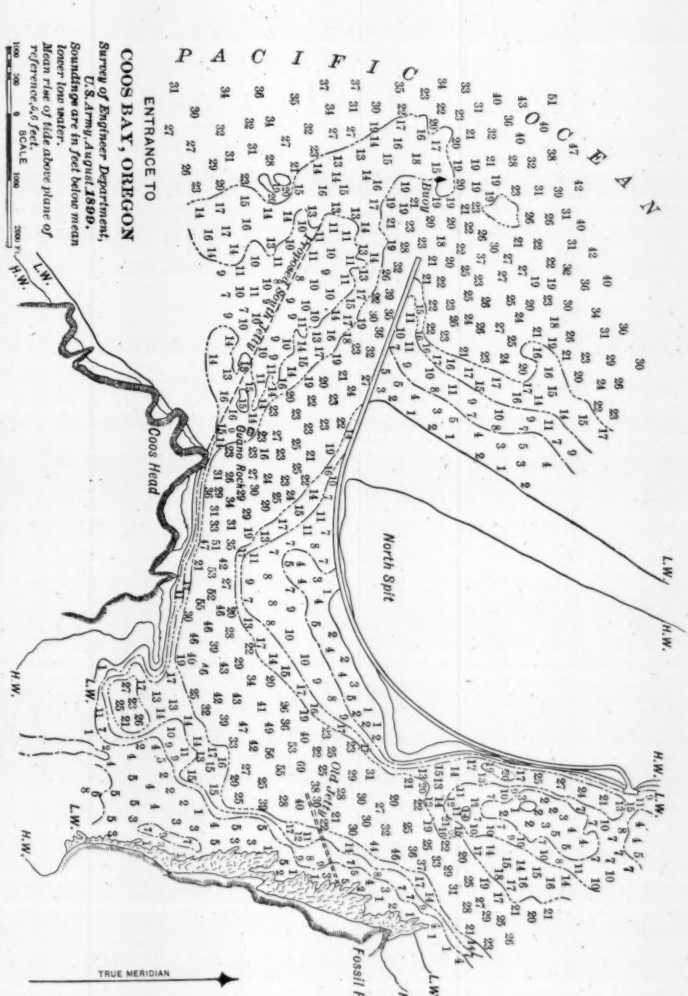

PACIFIC OCEAN
ENTRANCE TO
COOS BAY, OREGON
Survey of Engineer Department,
U.S. Army, August 1899.
Soundings are in feet below mean
lower low water.
Mean rise of tide above plane of
reference, 5.6 feet.
SCALE
Buoy
Proposed South Jetty
Guano Rock
Coos Head
North Spit
Old Jetty
Fossil Point
L.W.
H.W.
TRUE MERIDIAN

FIG. 5.

Dumping from both tracks continued until the mound was well above high-water line, an extra amount, to provide for beating down by waves, being placed at the outer end when that part was built. The slopes were those which the enrockment took naturally under wave action. Sections recently made at the outer end, where the worst wave action takes place, show that below a level of 15 ft. below mean lower low water the slope is about 1 on 1, but above this level the slopes were about 1 on 3. Two 15-ton derricks were used on the wharf in unloading, the stone being measured at first by water displacement of scows, but later by track scales. The latter method is by far the more satisfactory, both to the contractors and the Government.

The dump cars* were of a special pattern, designed by Mr. A. Seymour Fleet. They were arranged so that the car platform was removable, making a skip that could be loaded on the scow and then hoisted to place on the car. The skip when in place on the car was swung on horizontal trunnions so that when loaded the equilibrium was unstable, the skip and the load being held in position by a rack and pinion worked by a worm gear at one end of the car. This worm gear was arranged so that the stone when once started could be dumped on either side by turning the worm; the gear could be thrown out of action by removing a pin, allowing the stone to go by the run. The center of gravity of the empty car platform was so placed that its weight tended to return it to place. This design made it possible for one man to dump a train of six or seven cars in a few minutes. Cars of this pattern were used extensively on the Columbia River jetty, where they were likewise found very satisfactory.

Cost.—The entire plant used on this work, with its original cost, consisted of the following:

20	geared dump cars, at about	$800 each.	$16 000
2	Baldwin narrow-gauge locomotives, at	3 500 "	7 000
1	Fairbanks track scales		400
2	15-ton Lidgerwood hoisting engines and derricks, at	2 000 "	4 000
7	300-ton deck scows, at	2 000 "	14 000
1	towboat		10 000
1	revolving track pile-driver, entire		12 000
	Wharves, buildings and small tools, about		11 600
	Total		$75 000

*A view of the dump cars, loaded with rock, may be found in Mr. Symons' paper, "Jetty Harbors of the Pacific Coast," in *Transactions*, Am. Soc. C. E., Vol. xxviii, Plate xxiii.

The detailed cost of the work was approximately as follows:

Brush fascines	$1.75 to $2.25 per cord.
Poles	17½ cents each.
Piles delivered in rafts at the work	3¾ to 5½ cents per linear foot.
Rails, 30 lbs. per linear yard	$48 per ton delivered.
Common labor, $1.75 to $2.00 per day, and other labor in proportion.	

The jetty tramway, complete, but exclusive of superintendence or plant, cost $3.25 to $4.60 per linear foot, depending on the length of the piles used. The average cost was $3.60 per linear foot.

The average cost of the mattresses in place was 78 cents per cubic yard.

The average cost of the stone was 60 cents per ton, made up as follows:

Quarrying and loading on scows	38	cents per ton.
Towing to jetty	9	" "
Unloading and dumping in place	13	" "
Total	60	" "

The total cost of the jetty was about $70.30 per linear foot, the average quantity of stone per linear foot being 67 tons. The outer 1 000 ft. have taken much more than the average, 350 tons per linear foot for this distance being placed. This has been mainly to make up for subsidence caused by storms.

The average cost per linear foot shows very favorably when compared with others on this coast:

Yaquina Bay, cost	$106.20	per linear foot.
Coquille River, cost	70.63	" "
Columbia River, "	87.00	" "
Humbolt Bay, "	141.50	" "

It may be stated, however, that conditions for quarrying and transporting stone at Coos Bay are very favorable for this kind of work.

Results.—In extending the jetty, but little effect on the bar was noticed until the old swash channel to the north of the entrance was closed in 1892. From this time on, as the jetty was extended, the effect on the bar was more favorable with each extension. In 1893, the depth

desired, viz., 20 ft., at mean lower low water, was found on the bar, and from that date to the present time this has been the ruling depth. With one exception, in November, 1899, when the depth shoaled for several weeks to 9 ft., due to a long and severe southwesterly storm, a depth of from 18 to 22 ft. has been maintained for seven years. Thus far, this improvement has been eminently successful. Whether the good results already obtained will be permanent without further work in extending the north jetty, or whether the south jetty will be ultimately necessary, cannot be definitely foretold at present. It appears probable, however, from the most recent surveys, that the south jetty will be needed before the occasional lapping of the south spit around the entrance in southwest storms can be controlled.

The shore line has followed out, as the construction of the jetty proceeded, backing up the jetty with sand and thus strengthening the shore end of it, so that but little new work has been necessary on this part since the first work was done. This new shore line is from 1 200 to 2 000 ft. in advance of its original position. The large quantity of sand thus impounded is practically all saved from injuring the channels, and the work of the ebb tides in deepening the bar is thus made considerably less.

Experiments in holding this sand from drifting have been made by planting numerous kinds of trees, grasses and Scotch broom, and by using sand fences. The best results were obtained by using the Holland grass (*Arundo Arenaria*), which has been found so effective at the Golden Gate Park in San Francisco. This plant is of slow growth at first, but when it once takes root it grows with great persistency, catching and holding sand until oftentimes mounds 6 to 10 ft. high are made. Long patches, amounting in all to about 20 acres, have been newly set out, in the belief that it will bind the sand in the areas covered.

Remarks.—In reviewing the details of the construction, two points appear which are not yet sufficiently developed in this kind of construction: One is the value of brush mattress, and the other the means of maintaining to full height the outer ends of the jetties in a sea of such severe storms as are found in the Pacific Ocean.

The function of the mattress is to hold the bottom beyond the end of the enrockment during jetty construction and to prevent the scour which would otherwise naturally take place and thereby add to the

quantity of rock necessary. This holding action is probably secured by checking the currents and entangling in the twigs and meshes the sand borne by them. Mattresses also prevent the flow of currents, through the interstices in the enrockment, along the sand of the bottom. Such currents might, by long action, undermine the jetty; and, further, where the expected scour occurs in the channel, it is supposed that if any undermining is caused by racing currents too close to the jetty, the mattress, being flexible, will shape itself to the new bottom and stop further cutting. It has never been shown on this coast that these functions are more than partially performed.

It is believed that the mattresses, after being loaded with rock, are squeezed into very much less than their original volume, probably less than half. The comparatively high cost and difficulty of placing mattresses, and the time occupied, make it appear that, as volume in the completed jetty, the rock is considerably cheaper and better, being unaffected by sea worms. The plan of doing without mattresses altogether has been tried at a special place in the north jetty at Coquille River and in the jetty at Siuslaw River. The time is short since this was done, but no difference in settlement has been yet noticed. At Humbolt Bay, Cal., Lieutenant-Colonel W. H. Heuer, Corps of Engineers, abandoned brush mattresses in 1898, using a mattress of small stone as a foundation for the jetty enrockment at that place. This practice followed experiments of several years in both jetty and spur construction where mattresses of small stone were used with very satisfactory results.* Small rock can follow the tramway construction faster than brush mattress, and much scour around the piles is thus avoided in strong currents. The necessity and wisdom of using mattresses, especially in water more than 15 to 20 ft. deep, is questionable.

A weak point, thus far, in all jetty construction on this coast is the difficulty of maintaining the outer ends to a sufficient height. The heavy seas in storms break over the top of the enrockment and slowly batter down its height. The Columbia River jetty now requires considerable raising at its outer end to compensate for this action. The Humbolt Bay jetties are likewise beaten down at their outer ends. Coos Bay jetty has been twice built to 28 ft. above low water, and twice has been beaten down to below low water, and at the extreme end it was found that previous work, originally built well above high water,

* Annual Report, Chief of Engineers, U. S. Army, 1898, p. 2949.

had been beaten down to about 15 ft. below low water, this being apparently the limit below which the wave action is not especially destructive. The Coquille River jetties have been somewhat lowered at their outer ends; and at Yaquina Bay, where the jetties are more or less protected by an outlying reef, the subsidence has been unimportant, but even here there has been settlement at the outer ends from high to low-tide levels.

It seems plain, after watching the action of the waves at Coos Bay, that this beating down is progressive, commencing at the extreme end only. The jetty is never much damaged back of the outer end. The jetty being almost head on to the usual direction of the severest waves, the largest blocks, weighing 10 to 12 tons, are often picked up first with the advancing wave and then rolled or carried down the slope, with the fall of the water following. Experiments with concrete blocks weighing about 25 tons each were made during last year. These blocks were made in place at an exposed point to see what the action of the sea would be. Several were moved, and one was rolled by the waves down the enrockment to the low-water line. It appears, therefore, that blocks of this size would probably be too small for a permanent random covering for the enrockment. If some solid structure, strong enough to protect the outer end, could be built, it is thought the jetties might be maintained to effective dimensions much more easily. Such permanent end, of course, should not be placed until the proper length of the jetty had been ascertained.

Several methods are suggested by which these outer ends could be better held: One, by using for the enrockment a random covering of large concrete blocks, each weighing about 30 tons or more, for protecting the outer end of the jetty; and another, by constructing a monolith of concrete in bags resting on a broad rip-rap foundation brought up to the level of about 15 ft. below low water, the monolith topped above low water with concrete placed in moulds, the whole placed from a platform resting on iron screw-piles, which could be left to support and strengthen the structure afterward.

DISCUSSION.

Mr. Haupt.

LEWIS M. HAUPT, M. Am. Soc. C. E. (by letter).—This timely paper, following closely on an official report on the improvement of the Brunswick Bar, Ga., calls attention, in a general way, to the methods now pursued and the results attained in the Government practice, and also serves to illustrate the very divergent conclusions reached from similar data.

As the United States is assuming a position of ever increasing importance as a maritime nation, it is of primary importance that the bars obstructing her harbors be removed in such manner as to secure the best depths available at the least annual cost for maintenance.

These harbor problems, therefore, are of national importance, and their discussion, without prejudice, should result in the greatest possible economies and in the substitution of scientific methods for tentative experiments.

This paper is carefully prepared, gives evidence of much research and a clear comprehension of some of the difficulties, and, although mainly directed to the work at Coos Bay, it refers incidentally to several other localities, with conclusions which are so different from those reached by others who have studied the record that the writer takes the opportunity of adding a few extracts for the clearer presentation of all the physical conditions and results.

Taking up the various points in the paper *seriatim*, it is observed that the opening paragraph states that the completion of the north jetty has produced satisfactory results. The author then describes the exceptional violence of the storms of the North Pacific and the exposed condition of the harbor entrances, citing, but not applying, Stevenson's oft-quoted formula, $h = 1.5 \sqrt{d}$, where h is the height of the wave in feet, and d, the fetch in miles.

The danger of using a formula for such computations, in which the constant is fixed and empirical, will become apparent at once when it is considered that for d any distance may be used from 100 to 6 400 miles, and hence h will vary accordingly, so that the wave height may vary (using these values) from 15 to 120 ft. As the maximum waves seldom exceed 40 ft. in height it is evident that the formula has only a limited application, and must be used with caution.

The author next cites the early tentative works constructed within the coast line, which he remarks were only partially successful, but he gives no specific locations. The harbor under discussion, however, shows one instance of a wing-dike built out into the channel from Fossil Point (see Fig. 3) for a distance of 1 700 ft., but which he says, on page 496, had no useful effect on either the direction or depth of the bar channel, and was abandoned in 1889.

Mr. Haupt. There were interior jetties built on the New Jersey coast also, which likewise ended in failure and were abandoned. The single interior jetty at the mouth of Coquille River, Ore., in 1886, showed a low-water depth of 6.5 ft., now 6 ft.

It is almost an axiom in engineering that we learn more from our failures than from our successes; hence, it is of greater service to consider the former seriously and with no reflections upon those in charge of the works, since the actual condition of all the forces which may surround an entrance are occult, and, like those producing earthquakes where least expected, cannot be predicted.

The author very properly recognizes the importance of the resultant littoral movements, the cyclic oscillations of channel, the tidal action of flood and ebb in building and scouring bars, the necessity of conserving the full tidal prism, and especially, as he puts it:

"To prevent the encroachment of the sand on the channel and at the same time prevent the divergence and dissipation of the effect of the ebb tides, by confining and directing these tidal currents at a selected point of the bar."

He adds, if this can be done by suitable works then "improvement may be confidently expected."

In this he is entirely correct, but the difficulty which arises in their application, is in determining the actual position of the resultant and in so placing the works as to aid the land forces as opposed to the sea forces. One would suppose that, to protect a channel from the predominating drift, the barrier should be placed to the "windward," or between it and the channel, and hence it may excite surprise to find that most of the jetties which are built for this purpose have been located on the far or leeward side of the channel desired to be created, with the invariable result of rolling the bar rapidly seaward. In one case it advanced half a mile in about a year, adding greatly and unnecessarily to the cost of the work. But there are other conditions, in addition to these, which do not yet appear to be fully recognized and applied on ocean bars, one of which is to avoid an abrupt angular change of direction of the ebb currents which wastes their energy in boring deep holes, to no useful purpose. As this is to be merely a discussion of the paper, however, and is not designed to be suggestive of improved devices, it remains to show how the foregoing requirements of the author have been construed and applied in practice.

In the official Report on the Brunswick Bar,* where there are no jetties, and which was deepened 5.6 ft. by the use of dynamite, aided by dredging, it is said:

"This principle requires the jetty to be located upon that side of the channel toward which the latter is being driven by the drifting sands. To use a phraseology analogous to that which has been used in other cases, it should be on the leeward of the channel."

To make this still plainer as to the intention, it is added:

* Document, 355, H. R., 56th Cong., 2d Sess., Jan., 1901.

"A single jetty on this principle at Brunswick would be located on the south of the channel, since the drifting sands come from the north." Mr. Haupt

He then goes on to illustrate, by saying:

"A single jetty at Coos Bay, Oregon, has been built in accordance with this theory, and appears to have been successful in increasing the depth from 10 ft. to not less than 18 ft., which latter it has maintained for the last 5 years."

Here the north jetty is the only one built, while the paper states that the resultant drift comes from the south. It must, therefore, strike the jetty on its south or channel side, and hence the jetty does not fulfil the condition laid down by the author of this paper, viz., "to prevent the encroachment of the sand on the channel," while it does comply with the incorrect location suggested in the Report on the Brunswick Bar by being placed to the leeward of the channel with reference to the prevailing drift. Both of these theories cannot therefore be correct, and the case requires further examination to ascertain which is right. As the results are pronounced satisfactory by both authors it becomes necessary to look further into the records as to the original conditions, and to ascertain present effects on bar movements.

The depths prior to improvement are shown on the several charts of the paper, and which contain an admirable object lesson for the reader. In 1863 when the channel was flexed farthest north and was inside of the meridian through Guano Rock the ruling low-water depth was 7 ft. As it receded to the south in 1879 it is shown as being 13 ft.; again, in 1885, it is 8 ft., but farther out to sea; in 1894, with the jetty well advanced, it is 19 to 20 ft., with the 18-ft. contour south of but parallel to and beyond the jetty; and four years later, with the jetty completed, the depth is given as 19 ft. on the chart (18 to 22 ft. in the text), and it is 1 000 ft. farther seaward, with the 18-ft. contour overlapping the line of the jetty produced. During November, 1899, "the depth shoaled for several weeks to 9 ft., due to a long and severe southwesterly storm." The author, therefore, adds:

"It appears probable, however, from the most recent surveys, that the south jetty will be needed before the occasional lapping of the south spit around the entrance in southwest storms can be controlled."

In this respect the writer is pleased to agree with the author, for the reason that such a jetty will fulfil to the best advantage the main requirements of this inlet, but it can be placed much more effectively than on the broken line, as suggested on Figs. 4 and 5. This plan reduces the improvement to two high-tide jetties which have more serious objections than the author appears to attach to them, and which are worthy of consideration. The jetty now in place has apparently prevented the channel from drifting to the north where the energy was wasted and the sea more dangerous, being broadside; but the author has stated that at times, when the cycle was complete, the

Mr. Haupt. north channel closed and a new one opened abreast of the gorge of the harbor, when depths of 30 ft. at high water were reported. Deducting the tide of 5.6 ft. would give 24.4 ft. as the natural mean low-water depth over the bar, when the channel is in this position. So that the natural depths were, say, about 5 ft. better before improvement than those now existing. But the report of the Assistant Engineer in 1892, which appears to have been overlooked in all references to this work, states that the depth here prior to the beginning of the work was, at times, 27 ft. at mean low water, so that it would appear that for this position of the channel the unimproved depths should

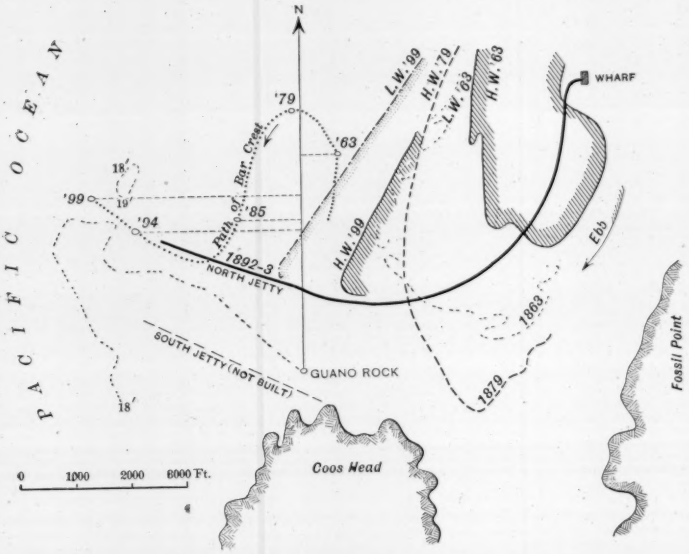


FIG. 6.

range from 24½ to 27 ft. instead of from 18 to 22 ft., or that there has been a loss of from 6½ to 5 ft., due to the effort to fix the location of the channel by a jetty placed to "leeward." Had it been placed to windward there is no doubt, in the writer's opinion, that the depths would have been greater and the bar advance less, with the further great advantage of acting as a breakwater for navigation, placing the vessel to leeward of the jetty instead of to windward in the great storms from the southwest.

The movements of the bar are perhaps more clearly illustrated by

plotting the location of the crest at the date of the several surveys and connecting them by a line showing the locus of this movement. As the actual crest is an area and not a point, this method is merely a guide, but it is relatively correct. The seaward movement is given here in departures from the meridian through Guano Rock as a well-defined fixed point, and hence they are not quite normal to the coast. Mr. Haupt.

The author states that the jetty "stopped on what was originally the 20-ft. contour * * * or just short of the bar proper," leaving it to be seen from the chart that the bar has advanced, which is a thing to be avoided. He does not appear to pay any particular attention to this feature, and hence the diagram, Fig. 6, will serve to show the rate and direction of this movement which is of great importance.

Table No. 2 gives the rate of movement for the intervals between surveys.

From Table No. 2 it appears that the bar crest has been advancing most rapidly seaward since the construction of the jetty, with every tendency to overlap it and ultimately repeat its cyclical peregrinations in a more exposed position. Under these conditions, can the work be classed as a success in its present transition state?

TABLE No. 2.

Survey of—	Period, years.	Total departure, in feet.	Average advance per annum, in feet.
1863–79	16	1 100	70
1879–85	6	1 200	200
1885–94	9	1 500	166*
1894–99	5	1 000	200
GENERAL AVERAGES OF ADVANCES.			
1863–99	36	4 800	133
1879–99	20	3 700	185
1885–99	14	2 500	150
1894–99	5	1 000	200

* Jetty partly built.

Two Jetties.—As to the effects upon the harbor from the use of two jetties, the author believes that:

"With double jetties properly designed it appears that no further effect is produced than would be caused by moving the throat or the gorge to a new point."

This conclusion does not seem to accord with observation, as it is recorded that double jetties have reduced the tidal inflow very seriously and caused a silting up of the harbors. To shift the throat so much further seaward is, in itself, an injury, as the resistance to discharge is increased by the longer channel; the entrance is partially

Mr. Haupt. dammed up and the area of the gorge on the bar site is also contracted. The present gorge shows depths at the point of incidence of ebb currents on Coos Head of 60 ft. in 1863; 61 ft. in 1879; 50 ft. in 1885; and, after the jetty was started in 1892, of 48 ft. in 1894; and 55 ft. in 1899. These are all reaction, not velocity, depths; an important distinction. An ebb current moving axially between nearly parallel jetties will cause no reaction, and no such depths at the outer throat can result as at the gorge, so that the outer cross-section will be smaller.

Engineers seem to have failed to note the fact that the deep holes at points of incidence or around bends of streams are not velocity depths, but are due to reaction from impact. For instance, in a report on a project for New York Harbor entrance, in 1885, it was proposed to transfer the throat at the Narrows to the outer bar by two rip-rap jetties rising only to low water, or below, and extending from Coney Island and Sandy Hook seaward, closing up the entrance almost entirely. The cost was estimated at $5 000 000 to $6 000 000. The precedent for this project was derived from the existing depths at the Narrows, for the report says:

"The flow through the Narrows * * * maintains a channel a mile wide and with depths of 100 ft. This depth only diminishes to 30 ft. at a point 4 miles below. The mean ordinary velocity at the Narrows is, during the ebb tide, about 2 ft. per second, and from this a depth of 100 ft. results."

The mistake made here is in attributing the depth to velocity, whereas it is due to the reaction caused by the "compression" (or, as some writers prefer to call it, concentration) of the flood tide in entering the Upper Bay. Many instances might be cited of mean velocities of 2 ft. per second flowing over sandy bottoms which do not erode to depths exceeding 4 to 6 ft. Hence it cannot be velocity alone that produces depths of 100 ft., but a much more potent force, viz., impact creating reaction from the bottom.

The effort, therefore, to utilize the gorge depths by reproducing them at the bar end of a pair of jetties has invariably resulted in failure, and still it is recommended to be tried over and over again. The New York project was soon abandoned. The latest instance is found in the Report on the Brunswick Bar, already cited, in which it is said:

"Apparently, if all the ebb tide is concentrated between the jetties on the bar, and their width apart is equal to that of the gorge, the conditions of the latter would be reproduced on the bar. This would require jetties to high water."

The author then goes on to explain that as such depths would be excessive and the cost greater, only low-water jetties are usually built, and adds:

"With this system of construction, however, it has been found that the scouring effect has generally not only failed to reproduce the

depths at the gorge but has not produced depths enough for navigation, * * * (still) the channel location has been fixed so that very satisfactory dredged channels can be maintained," and the experience at Charleston is given as "a good example." Mr. Haupt.

It would seem, that if high jetties would produce "depths far beyond the requirements of navigation," and if low-water jetties having "the width between jetties on the bar generally greater than at the gorge" would "not produce depths great enough for navigation," that between these limits there should be found a happy medium, at which the depths, resulting from tidal scour, should suffice without recourse to dredging.

If Charleston is cited as a good example of dredging, supplemented by jetties, or the reverse, let it be observed that the bar is now about 4 000 ft. beyond the end of the completed jetties, and the original channel was abandoned because of its rapid filling up. Moreover, the jetties are not low-water, but submerged, jetties 15 ft. or more at the inner ends and high-tide jetties over the original site of the bar, so that the reference is hardly germane.

Yaquina Bay.—Yaquina Bay has been referred to in recent publications as an illustration of the success of two high-tide jetties, and the present paper seems to confirm this opinion to some extent. The author says the south jetty did not accomplish all that was expected of it, and it was found necessary to close the old north channel "before much improvement on the bar resulted," so that it is an inference.

This problem and its solution cannot be fully understood without the plan, for it was complicated by dangerous submerged reefs lying off shore which served to create a lee for navigation, and any advance of the bar seaward as the result of two jetties was sought to be avoided by a single south jetty at an acute angle to the shore line, to close, in whole or part, the south channel. This project, as designed (see Figs. 7 and 8), is described in the report of the Chief of Engineers for 1881, page 2592, in these words:

"The existing plan of improvement provides for a jetty on the south 2 500 ft. long, rising 2 ft. above mean low water," to be built of stone and timber cribs. "Suitable appropriations should be made for an improvement which promises such good results."

The ideas intended to be applied here were to prevent the sand encroachments on the channel and the dissipation of the ebb currents, but it is unfortunate that the means were insufficient to produce the result, and the north jetty became a necessity, so that the original idea was radically changed to one of concentration between two jetties with consequent bar advances, which it was desired to avoid. This is now cited in the Report on the Brunswick Bar (p. 37) as one of the three "only examples of such high-tide jetties in the United States." The other two mentioned are at Sabine Pass and Galveston, both of

Mr. Haupt. which were originally designed to be submerged on their flanks, were subsequently raised to low water and finally to high water to control the sand movements and currents. There are, however, various other high-tide jetties in the United States, which teach the same lesson of congestion of tides and bar advance. While the first jetty, as designed, was being built at Yaquina, under great difficulty and exposure, it was barely awash at low-water, and formed a sharp re-entrant with the shore line. The breakers cut it off from the shore by a breach of 375 ft. width at the inner end, in 1883–4, which was repaired the next year.

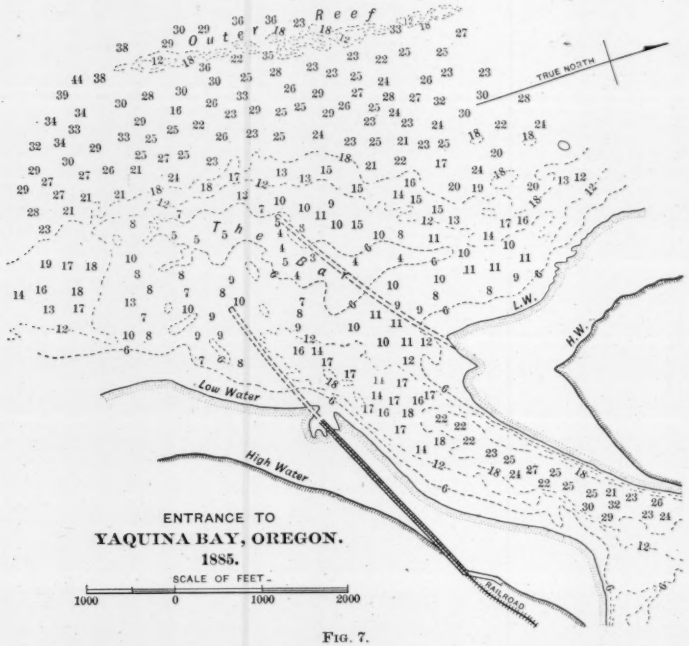


FIG. 7.

In 1886 it was recommended to extend it 1 000 ft., and to build a half-tide north jetty of 1 800 ft., with further extensions of both jetties, the north one 1 200 and the south one 700 ft.; at a cost of $800 000, "in round numbers," if ultimately built to the 18-ft. curve.

The minimum depths reported in the north and south channels were originally 7 to 9 ft. In the central channel at certain seasons there were from 5 to 10 ft. mean low water, while the mean tide is 7.1 ft. The proposed depths are referred to mean high water and were to

Mr. Haupt.

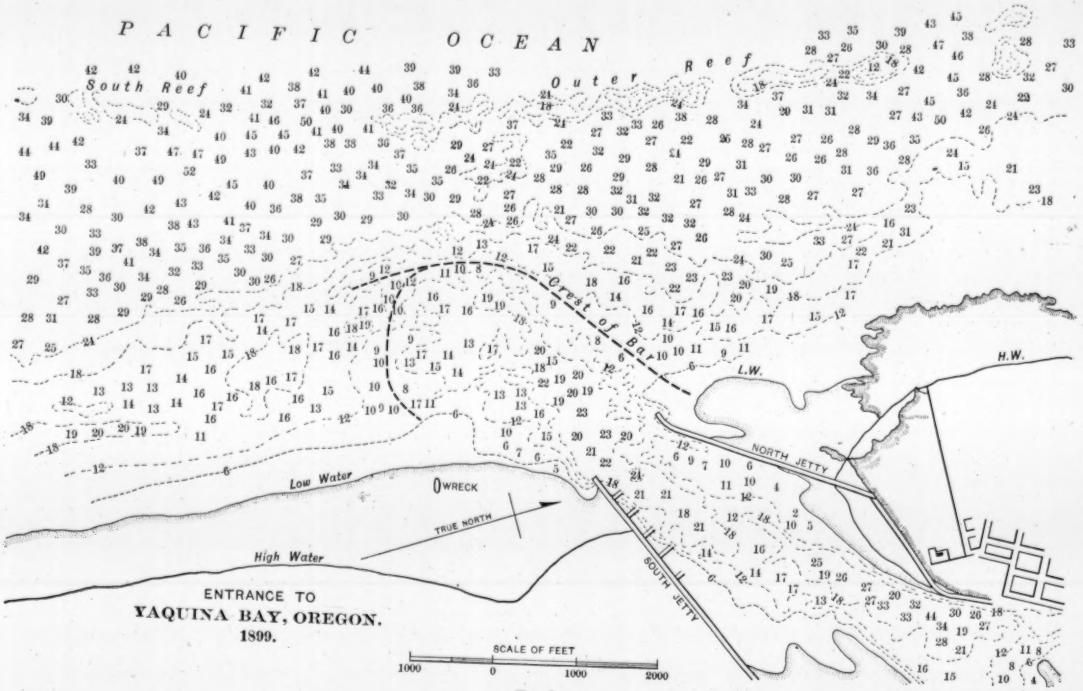


FIG. 8.

Mr. Haupt. be 17 ft. at first, afterward changed to 19 ft., so that the actual increase above the natural depths existing in the central channel at times would vary, when referred to the same datum, from zero to 2 ft. The report of 1886 contains a map dated 1885 (Fig. 7), which shows that the original 2 500 ft. of south jetty is surrounded by sand, as the low-water line has advanced beyond its end, and the depths in the south channel are still 7 ft., and in the north channel 8 ft., while an outer shoal of only 4 to 5 ft. depth, and 1 500 ft. distant covers the entrance. This report states:

"The Yaquina bar is narrow and near land, making its crossing easier than if wide and more seaward. * * * The main sand movement here is from the south. * * * The outer reef is a natural breakwater and reduces the seas on the bar, a most favorable circumstance.

"A proposed north jetty location and south jetty extension are shown on the survey map of 1885 herewith. The north jetty is 3 000 ft. long and projects beyond the south jetty from which it is 1 000 ft. distant."

Up to this date, therefore, the effect appears to have been an advance of the shore line and bar without any increase in depth due to this south jetty, which is stranded between high and low water, although the report of 1887 says: "The new depth is greater by 1 ft. than the best heretofore observed during the same seasons."

Operations on this jetty were resumed in 1886–7, and during the next fiscal year it was extended only 460 ft. In 1888–9 the south jetty was reported as having been extended 4 265 ft., and the tramway started for the north jetty, on which 12 980 tons were placed in 1889–90, and 18 040 tons on the south jetty. In 1890–91 the north jetty was extended 600 ft. In 1891–92 a large part of the south jetty was completed, and the north jetty was extended 840 ft.

In 1900 it was reported that the project of 1881 was practically completed in 1895, comprising the construction of a north jetty 2 800 ft. long and a south jetty 3 748 ft. long.

"It was expected that this work would obtain a depth of 17 ft. at high water, but the results are now somewhat in excess of that depth.

"There is at present, about one-half mile from the end of the jetties, a cresent-shaped bar almost inclosing the entrance and having from 8 to 12 ft. of water over it at low low water. Around the south end of this shoal there is a channel having a least depth of 13 ft. at low tide. This bar has apparently shoaled somewhat since the survey of 1895."

The map accompanying this report shows a limiting depth on the bar of only 10 ft. at mean low water, and, as the original depths were at times from 5 to 10 ft., the maximum depths of 17 ft. at high water have apparently not been increased materially, if at all, as the report admits, while the bar has been moving out toward the reef at least a quarter of a mile.

This project of 1881 was no sooner reported as completed in 1895

than another Board was appointed to investigate and report "with a view to a project for deeper water." It reported that, "if any further deepening of water could be effected it would only be by prolonging the present jetties," and it proposed to extend the north jetty 2 000 and the south jetty 2 200 ft.,* and to remove some detached rocks at the entrance, at a total cost of $1 025 800. Congress made a preliminary appropriation under this project in June, 1896, but the work has not been commenced and the Board of 1899 does not recommend anything further than the removal of the rock. The total cost of this experiment between 1880 and 1896 was about $688 000, and, in view of the history and results as reported, it is difficult for the writer to discover any improvement as to increased depths over the bar or even the fixing of the channel. Mr. Haupt.

The low-tide single jetty of 1881 was amended in 1888:

"To raise it to full high water with a length of 3 748 ft. and to construct a mid-tide jetty on the north side to extend in a southwesterly direction about 2 300 ft. and to give an entrance width of 1 000 ft. This plan was further modified in 1892 to raise the north jetty to high-water level and to build five spurs on the channel side of the south jetty to prevent undermining."†

The depths as reported in 1896, were said to have been increased to from 12 to 15 ft. at mean low water, and the project was considered completed.

As the reported depths in 1900 were a bar with from 8 to 12 ft., at low water and a channel depth of 10 ft., it would appear that, as the Board has reported, the bar has shoaled since 1895, and that the claim made that: "These jetties at Yaquina Bay have increased the depth from 7 to 15 ft. at mean low water, and, so far as the records show, without as yet producing any new bar seaward of the jetties," does not accord with the facts as officially reported by the Board of Engineers of 1900.

Considerable space has been devoted to this case, as it is believed to furnish no new evidence of the success of high-tide twin jetties, and it is respectfully suggested that the claim of the author that "for more than seven years, depths considerably in excess of those stated for the project have been maintained," might be modified to advantage. The projected depths, as revised, were 19 ft. at high water. The present map of the survey of 1899 (Fig. 8), made in June and July, if correct, shows the best channel depth to be 10 ft., to which, if the mean height of the tide be added, there results, 17.1 ft., or nearly 2 ft. less than those stated in the project. Moreover, the condition here is apparently not stable, but retrograding; and jetty extensions were recommended to improve the depths, but not applied, as it was thought the commerce did not justify the expenditure at this time.

* Report of 1896, p. 3242.
† Report of 1896, p. 3241.

Mr. Haupt. The author next refers to Coos Bay, Columbia River and Grays Harbor as illustrations of sites where "the natural conditions are such that one jetty has already accomplished so much improvement that the necessity for a second jetty is not yet apparent." It has been shown, however, that at Coos Head and Columbia River the bars have advanced, and the depths diminished. That Cape Disappointment is regarded as performing the function of a second jetty is admitted, while at Grays Harbor the partial jetty is located to leeward of the channel and gives no better assurance of bar control than the others. It was sufficiently demonstrated by the writer in the discussion of a previous paper,* and need not be repeated here.

Dredging Tidal Entrances.—One other point of importance is raised in this interesting paper. It refers to the methods of improving ocean bars by dredging, and the writer is pleased to find himself in thorough accord with the author when he says :

"As the forces going to build up the bars are intermittent, but constantly recurring, no temporary expedient, such as dredging, would be of any special value, owing to the extreme likelihood of such work being completely nullified by the effect of a single severe storm."

This is confirmed by repeated experiences on exposed bars.

On the other hand, after reviewing all the several methods of bar improvement, the author of the Report on the Brunswick Bar reaches the conclusion that, for that place, "dredging appears to be by all means the most economical and satisfactory method."

This is at an ocean bar about 4 miles from land and on the borders of the trade winds where the sea is frequently rough, and, while the cost per yard of the material removed is based upon "dredging," the author of that report says it "has been successful as to permanence of the improvement." He appears to have ignored the general disintegrating effects of the large charges (100 lbs.) of dynamite which were used on this bar simultaneously with dredging. It becomes, therefore, an important matter to study the relative permanency and depths secured as the work progressed, the data for which are furnished by the very complete exhibits contained in Doc. 355, before quoted.

Movements of the Thalweg of the Brunswick Bar Channel.—First, as to the original condition, the report says :

"Prior to 1879 the channel depth across this bar was about 17 ft. at mean low water. Between 1879 and 1889 this channel became obliterated, and the best navigable depth at the latter date was about 13 ft. at mean low water. The present navigable depth is 18 ft. at mean low water."

Speaking of the changes in location of the channel, the report says :

"Brunswick Bar is an exceedingly slow one to change. The reason of this is that it is not in a region of long-continued storms or high winds, etc." (page 4).

* *Transactions*, Am. Soc. C. E., Vol. xxxvi, p. 133.

Mr. Haupt.

The experience of the Coast Survey Assistant who, in making a survey, was detained on the bar from December 27th until January 4th, "before the weather permitted a beginning to be made on the work," and the stormy and unfavorable season following, which still further prolonged the work "much beyond the date anticipated for its completion," may be taken as a fair type of the weather at that locality during the winter and spring months. The survey was not finished until April 16th, 1897. It was called for in the previous November, a period of six months.

The improvement of this bar began in 1891 by the use of dynamite, when the best navigable depths were "about 13 ft." Since that time, and up to the end of the survey made in November, 1899, the increase was 5.6 ft., and the cost per foot of depth gained was $45 293.95.* But the author of this report says:

"The channel now existing at Brunswick Bar has been produced by Nature, aided very materially by ordinary dredging. The useful effect of dynamite has been very small, except possibly to loosen material in some places as an aid to dredging. The cost to the Government of all material removed, whether usefully removed or otherwise, has been $1.13 a yard, more than seven times the cost of ordinary dredging. The channel produced has no advantage of permanence over a dredged channel of similar size and location which could be produced for about one-twelfth of the cost of the present channel."

By "ordinary dredging" here is meant the use of a hydraulic plant with bins.

An analysis of the foregoing conclusions shows that, according to the author, Nature and dredging allied have produced the present channel, upon which the effect of the dynamite "has been very small," yet the channel thus produced by dredging and Nature has no advantage of permanence over a dredged channel which could have been produced for about one-twelfth its cost. Hence, it would follow that the work done by Nature alone must have added eleven-twelfths, or about 92%, to the cost. Therefore, the engineer should aim to eliminate any assistance from Nature, and confine himself to mechanical means to cheapen the works.

As to the permanence of this channel, it is here stated that it has no advantage over a dredged channel, while on another page of the same report it is said of this bar:

"It is an enormous sand bank which moves and which always moves very positively in one direction."

This being the case, an unprotected, dredged channel would naturally be expected to fill up in a comparatively short time unless protected by a barrier; but the report says of this dynamited channel:

"It has been quite successful, as to the permanence of the improvement."

From the maps and table on page 11 of the report quoted the data in Table No. 3 have been collated:

* Senate Doc. 178, 56th Cong., 1st Sess.

Mr. Haupt.

TABLE No. 3.

Map.	Date.	Interval, in months.	Depth, M. L. W.	Gain or loss.	Movement, in feet.	Direction of motion.	Form of channel.	Notes.
A	Jan., 1891	..	13.3	...	000	.....	( )	Lt. Rees, U. S. E.
B	April, 1891	3	13.4	0.1	800	South	(	Dynamite began in July and Aug.—1 650 total.
C	Sept., "	5	13.4	0.0	1 150	"	(	
D	June, 1892	9	13.7	0.3	500	North	) (	Hughes, U. S. N.
E	Nov., "	5	14.0	0.3	350	"	))	Rees, U. S. E.
F	June, 1893	7	14.2	0.2	100	South	))	" "
G	Dec., "	6	14.2	0.0	200	"	))	Carter's Assist., U. S. E.
H	March, 1894	3	13.9	0.3	225	"	))	Garret, U. S. N.
I	April, 1895	13	15.0	1.1	250	"	))	Abbot, U. S. E.
J	July, "	3	14.9	0.1	300	"	))	" "
K	Dec., "	5	15.0	0.0	325	"	))	" "
L	April, 1897	16	16.6	1.6	500	"	))	Dredg. without bins, Peck. U. S. C. S.
M	Feb., 1898	10	17.6	1.0	600	"	( )	Ordinary dredging. Marindin "
N	April, 1899	14	17.8	...	950	"	( )	" "
O	Nov., 1899	..	18.5	...	1 000	"	( )	" "

Several features are prominently brought out by this exhibit:

1.—The progressive deepening, increasing more rapidly as the channel becomes more stable.

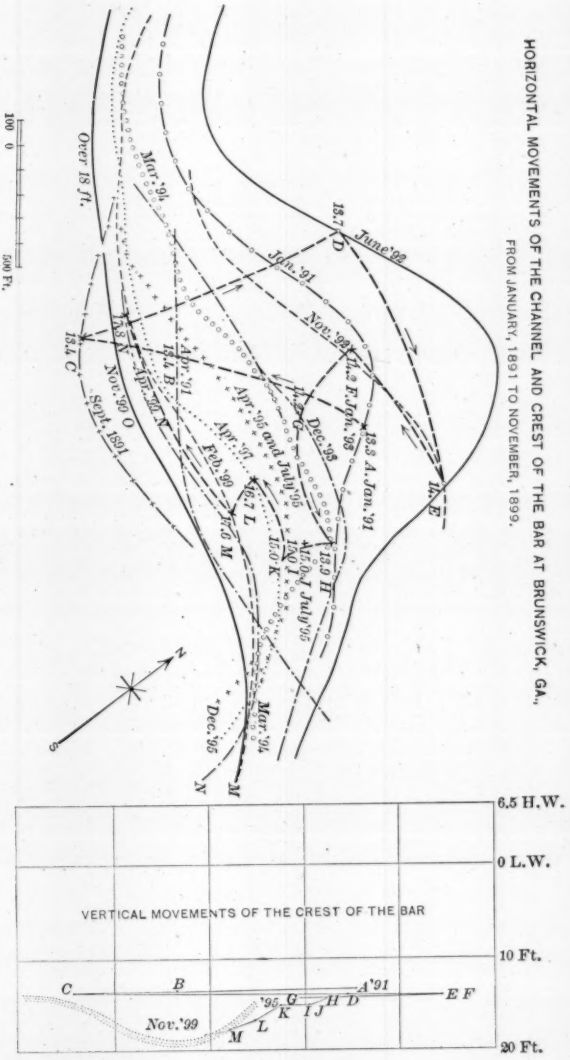
2.—That at first the bar crest was so nearly level and so close to the ordinary limit of wave disturbance that a few inches of change in its elevation caused great changes in the location of the channel.

3.—That dredging did not begin until between December and April, 1897, and then it was merely to stir up, but not carry off, the silt. At this date the depths had increased from 13.3 to 16.6 ft., and, by the time the hydraulic dredging and removal commenced, the depth (by interpolation) was about 17.2 ft., showing a gain, due almost entirely to dynamiting, of 3.9 ft. The subsequent increase to November, 1899, is 1.3 ft., but even this was not due to dredging alone, as the report shows that dynamite was used throughout the entire period, at first in 10 and 20-lb. charges, and after August, 1892, in 100-lb. charges.

4.—The greatest change in the position of the channel occurred between September, 1891, and June, 1892, nine months, when it moved 1 650 ft. from its extreme southerly to its extreme northerly limit, due to a change of level of only a few inches on the bar.

5.—From June, 1892, it steadily drifted back to the south, and as the depths increased the movements decreased, while the thalweg became more direct.

6.—The path of the channel is never straight, but always curved; more sharply in the shoaler depths and more gradually as the depths increased. It now assumes the reverse-curve form adopted for the reaction breakwater.


HORIZONTAL MOVEMENTS OF THE CHANNEL AND CREST OF THE BAR AT BRUNSWICK, GA.
FROM JANUARY, 1891 TO NOVEMBER, 1899.
100 0 500 Ft.
Over 18 ft.
Mar. '94
June '92
July '91
Nov. '99
13.7 D
13.4 C
13.4 B
13.4 A
Nov. '99 O
Sept., 1891
Apr. '91
Apr. '95 and July '95
Dec. '98
13.3 F. July '98
13.3 A. Jan. '91
14. E
Feb. '99
16.7 L
16.6 M
16.0 K
16.0 I
15.9 J July '95
13.9 H
Mar. '94
Dec. '95
M
N
S

6.5 H.W.
0 L.W.
VERTICAL MOVEMENTS OF THE CREST OF THE BAR
10 Ft.
20 Ft.
C
B
A '91
E F
'95
G
H
D
K
I
J
L
M
Nov. '99

FIG. 9.

Mr. Haupt.

7.—The ratio of depth before hydraulic dredging to that after is 61%, although dynamite was used after as well as before, so that the effort to make it appear that the channel was secured by Nature, aided by dredging, is, to say the least, misleading.

8.—The breadth across the bar is about ½ mile, and it remains practically constant, while the points of ingress and egress are not shifting. The bar, therefore, has not advanced seaward, although the location of the point of least depth has been changing as the depth increased.

The effort of Nature is to fill up the channel by this traveling bank "which moves very positively in one direction," as the surveys also show. The cross-section also confirms this statement, as the report says:

The present channel "is about 300 ft. wide, and the side slopes on the south are about 1 ft. in 100. The north slope is steeper—about 1 ft. in 30."

These slopes indicate the direction of movement of the bank to be southerly. (See Fig. 9.)

The form and permanency of the channel, therefore, must be attributed to the use of dynamite in enabling the currents to select, create and maintain a channel along the line of least resistance, and they all tend to demonstrate that had a straight channel been dredged across this bar at any point it would not have been adapted to the natural conditions and could not have been self-maintaining. This is the general experience elsewhere.

But there is another exhibit in this report which is of greater importance, because, from it the estimated cost ($1.13 per cubic yard) of material removed was derived.

It is given on page 14, and purports to show the volume of scour and deposit within certain areas, in the vicinity of and including the channel, for strips varying in width from 400 to 2 000 ft., and for the years from April, 1891, to November, 1899. From this it appears that for the following widths the quantities for a length of 4 680 ft. are:

400 ft. wide	.....	166 892	net cubic yards.
600 "	.....	209 598	" "
800 "	.....	218 260	" "
1 000 "	.....	191 420	" "
1 200 "	.....	176 000	" "

From this it appears that the strip 800 ft. in width shows the greatest net gain, and as the total cost, as stated in Senate Doc. 178, 56th Cong., 1st Session, is $253 646.15, this, at $1.13 per yard would require the removal of 224 465 cu. yds., so that the estimate is apparently based upon the net displacement of material during the entire term. But the author says that a channel as good as the present one could have been secured by the dredging of 125 000 cu. yds. at a total cost of $18 750.

If so, is it not a severe reflection upon his predecessors that it has never been done, either here or anywhere else? Mr. Haupt.

The error of this calculation arises from considering the quantity of material in a given area as a constant, and assuming that the portion excavated from the channel is in part, at least, deposited over its flanks, constituting merely a change of form without material change of volume. It produces the anomalous results, as shown in the first part of Table No. 3, viz., that for every width considered in a strip of 6 370 ft. in length, between the years 1891 and 1897, there was a net fill varying from 76 733 to 208 416 cu. yds., and yet in this time the channel was shifting, earlier ones being filled and later ones opening, with a gain in depth of channel of 3.3 ft. On this basis of payment, the contractor would be a debtor instead of a creditor.

It is, therefore, unjust and inaccurate to measure quantities in place, where a bar is in motion and when superfluous material is being constantly supplied to replace that which has been removed.

The writer knows of instances of dredged cuts having refilled within a few months of the completion of the contract. Had the material been measured in place instead of in the scow, the contractor would have been ruined, and so must everyone be who undertakes to base his bid on net measurements before and after execution.

It follows, therefore, that any conclusions based upon the cost of dredging on a moving ocean bar at even 15 cents per yard, with the hope of securing and maintaining a permanent channel, must end in disappointment and failure. The record of experience is no doubt of more value than many opinions, and that shows that the cost of obtaining depths by any of the existing auxiliary methods is about ten-fold that of the method pursued at Brunswick, for each foot of depth actually secured.

This demonstration of the use of dynamite as an agent in assisting Nature has, therefore, proved to be remarkably successful at the Brunswick Bar, which has not shoaled materially in the past two years, during which no work has been done. A careful analysis of the various methods used in the United States during the past thirty years shows that they have not fulfilled the conditions laid down for them.

The two-jetty plan of submerged jetties as designed for tidal estuaries has been abandoned, and at Charleston, where they were constructed at a cost of about $4 000 000, the fallacy of admitting the sand over the flanks only to be carried seaward at ebb to supply the bar is fully demonstrated. Low-tide jetties have been raised to mid-tide and subsequently to high water to control the sand movements and defend the channel, but, as a rule, the foreshore advances and the bars still move seaward retaining the same depths except as they are increased temporarily by dredging at large annual cost for maintenance;

Mr. Haupt. while the tidal energy is reduced by the partial dams built on the flanks of the entrances.

When single jetties have been attempted they have been constructed, almost invariably, first on the wrong side of the channel, where they have created deposits in it and a more rapid advance of the bar seaward; and, ultimately, a second jetty has been required to restrain the drift, but after it was too late and the original entrance was closed, as at Cumberland Sound. These expensive experiments, of which there are so many on record, do not seem to have led to any modifications in practice, for still it is recommended to build the single jetty on the leeward side of the channel, where it must dam it up; and the method of tentative experimentation goes merrily on.

In the discussion of this valuable paper, Captain Black says truly that hindsight is better than foresight, but in taking his bearings upon the retrospect he seems to have gotten on the wrong trail, which would lead to some erroneous deductions.

First.—The first of these is that although certain works were designed thirty years ago by officers, now deceased, it is found better to "complete them on the original lines rather than make them conform more closely to the plans which more recent experience has shown to be best." Most engineers, on finding the line too expensive and not well adapted to its purpose, would have changed the location for the best and most economical by taking a new "departure" and getting out of the rut.

Second.—Attention is confined chiefly to the movements of sand and the causes producing them, while the tidal volume and other important factors are ignored. No correct conclusions can be reached unless all the elements are considered, and they are many and variable. They have been set forth already in previous publications.

In describing the formation of the "crescent-shaped bar," it is said that it is "pushed out until the ebb currents have a chance to spread and lose their force," and "the ebb has joined to it the drainage of the area of the interior." In many cases the interior drainage is so small that it is less than sufficient to provide for the evaporation in the large interior lagoons or bays receiving it. It is therefore negative, and hence the entire maintenance of the channel is made to depend upon the tidal energy, regulated by protecting works; and, as the bar itself is the location where the ebb currents spread the most and fail to maintain their velocity, that is where the assistance should be applied "to prevent the water of the ebb from spreading and losing its force." It has been successfully done by local work, which does not interfere with the ingress of the tide, upon which alone the results are dependent. The jetties need not be continuous from the shore out across the bar in tidal inlets. Such construction has invariably been found to be unsatisfactory. Captain Black's remarks are apparently confined to those cases

where there is a river emptying directly into the sea, in which cases a continuous single jetty, properly located, would be most effective, because the material of the bar is derived mainly from within, and not from the littoral drift. This river sediment is vastly greater in many instances than that from outside, and, if two jetties are built, the deposits must occur either beyond them, in the sea, or above them, in the river, thus raising its bed and its flood plane; in either case producing injury to the navigable channel, and requiring constant dredging. See the records of the South Pass of the Mississippi River. Mr. Haupt.

Third.—Attention is called to the unreliability of maps, as representing general conditions, because of the frequent changes produced by storms. But, as the map is supposed to be the record of the facts, at the time of the survey, it is the only reliable basis for study and deductions, and when a comparison of a number of maps shows a prevailing direction of movement it is certainly a reasonable inference that such representation is correct.

We must not discredit our data, else we have no basis for our deductions. The large number of surveys, by different authorities and under variable conditions, at Brunswick Bar, all showed progressive deepening during the use of dynamite and none before it; hence, the deduction is justifiable that the effect was due to that cause, though there are engineers who assert that it was not, but that "Nature" had come to the rescue. How or why, they do not attempt to explain, but rely on the simple negation.

If maps do not represent the general conditions, then, too strenuous an interpretation should not be placed upon them in deciding upon the fulfillment of a contract, and especially when weather conditions are such that the surveys cannot be completed until about six months have elapsed after the required depths were secured. In consequence, the shoaling of a few inches, in places, retarded payments for more than a year, although the terms of the contract were substantially and honestly complied with, and the channel had been in use by vessels of the requisite draft.

Fourth.—In considering the location of a single jetty on the South Atlantic, Captain Black concludes "that one jetty will rarely do its work," but he cites no actual precedent. He theorizes upon imaginary conditions, and does not realize the ultimate resources of the engineer in arresting littoral drift before it ever reaches the jetty, should it be necessary. He assumes also that "there is nothing to prevent the spreading before the working point is reached," forgetting that a current ejected from a coast between walls of water maintains its energy for a considerable distance, and that there is frequently a large excess, so that the utilization of only a portion of it may suffice for navigation purposes across the bar. After considering the case of a jetty on the north side to impound the sand, and then on the "south, concave to

Mr. Haupt. the current, in order to hold it," the conclusion is reached that "in both cases results are disappointing," as "could be illustrated from the history of both the St. Johns River and Galveston improvements."

While it is true that in both these instances the results were disappointing in the extreme, the hindsight is again defective, as in neither case was a single concave jetty built, but, on the contrary, in both cases two convergent straight jetties were constructed; hence, no deductions from them can be applied to a concave jetty located on the correct side of the channel. They have served their purpose in teaching what to avoid in future. These disappointing results were anticipated and reiterated by some of the civil engineers employed on the works, who were rewarded by being dismissed.

A continuous concave jetty, however, was built at Aransas Pass by the Government, was an acknowledged failure, and was abandoned; but a detached, concave breakwater built on the bar by private parties was a pronounced success, although only partially completed. What constituted this difference between success and failure at the same inlet—and at one-half the cost?

Mr. Black. W. M. BLACK, M. Am. Soc. C. E.—As age increases, the engineer, of all men, can bear witness the more from his heart that hindsight is much more sure than foresight.

In considering the Charleston and other jetties on the Atlantic Coast, mentioned by Professor Haupt in his comprehensive discussion on this interesting paper, it should be remembered that they were designed about thirty years ago, and that it has been found advantageous to complete them on the original lines rather than make them conform more closely to the plans which more recent experience has shown to be best.

Much has been written recently concerning the efficiency of jetties on ocean bars. It appears to the speaker, however, that in the discussions there are several principles which are frequently lost sight of. Let us take, first of all, the sand movement. What produces the bar in the first place? The sand is not banked up opposite the mouth of a river originally; it comes from somewhere. On the South Atlantic Coast it is known that there is a strong and almost continuous sand movement. This has been shown conclusively at the mouth of the Cape Fear River, N. C., and at the mouth of St. Johns River, Fla., for example. At the mouth of the St. Johns River, at one stage of the jetty construction, a great deal of sand was impounded on the north side of the jetties, and a portion of it passed around the jetties and was impounded on the south side. A summer resort had been located at Pablo Beach, about 6 miles below, and the result of this impounding of the sand at the jetties was an erosion of the coast at Pablo Beach to an extent sufficient to cause alarm to the cottagers. As soon as the sand had accumulated in quantity sufficient to fill the basins

formed by the jetties, the movement along the coast was resumed and the erosion at Pablo stopped. Mr. Black.

Along all our flat Atlantic Coast, sea bathers know that after a storm the foreshore for some distance out is in furrows. After a storm a beach which has been perfectly flat before the storm shows a number of deep channels running parallel to the coast. During a northeast gale, the waves approach the shore from the northeast on a line oblique to the coast. As they reach shoal water they are wheeled, as it were, partly parallel to the coast, but never entirely so. As soon as a wave gets into water of which the depth is about twice its height, the bottom of the wave is stopped; a portion of its energy is transformed into a motion of translation of the water; the bottom of the ocean is stirred up and the sand is carried ashore. When the wave finally breaks, more sand is stirred up. The sand is carried upon the beach with the wave wash which moves obliquely up the beach, turns and runs back, not opposite the point at which the wave struck, but further south. At the same time the whole of the ocean is in a measure backed up against the shore. The level is actually raised. In the harbors on this coast the flood tides are frequently made much higher, and sometimes the ebb is entirely imperceptible for twenty-four hours.

These two causes form, during every northeast gale, a very strong current close inshore, running in the direction of the line of least resistance, which, under a northeast gale, is generally to the south, south of Central New Jersey. The sand eroded in one place is ordinarily replaced by that eroded to the north of it; but if any part of it is impounded, the invariable result is to make evident the cutting below. In making groins for building out the coast, the first result is always a cutting—on the South Atlantic Coast to the south of the groin, with a fill on the north of the groin. As the fill increases, the sand on the north works around to the south. This sand movement is the cause of the wind and wave bars, and it is natural to suppose that that action does not cease, but is continually recurring. The height of the bar is a function of the currents and of the storms. When it gets too high the top is swept off and carried along; when too low it is filled up. At some places that movement is very rapid; at other places it is very slow; so that the condition even of sand bars in our harbors may be very stable or very unstable, and whether it is stable or unstable can only be determined by a study of the local conditions. At Pensacola, for instance, the bar is of heavy, coarse sand, and is quite stable. At the mouth of the St. Johns River, and the mouth of the Cumberland Sound, the bar is of very light sand and is very unstable.

Opposite an opening in the coast there is a deflection of what may be termed the sand current, seaward, because the ebb has joined to it

Mr. Black. the drainage of the area of the interior, and leaves the coast line, directed by the banks of the interior waterway, as a definite force. As a result, the deep-water channel of the interior waterway is conserved for a longer or shorter distance seaward from the shore line, and the sand flow along the coast is pushed out, making the crescent-shaped bar of which so much has been said—pushed out until the ebb currents have a chance to spread and lose their force.

Now, in a bar of that kind, it is evident that there are two distinct parts in the work of its improvement. The first is to cut through the obstructing bar; the second, entirely distinct from the first, is to keep it cut through; for it is evident that the forces which formed that bar first will work to maintain it. It is also evident that that movement along the coast, or in the direction of the resultant storm action, only extends to a certain depth, and that sand can be piled up to that depth with a certain degree of stability. As soon as this depth is decreased the wave and current action is bound to move the deposit along in exactly the same way as before. As stated, that depth is a function of the wave height. On the Pacific Coast the shores are very abrupt and the waves are high—as high as 30 ft.—and breaking in 6 to 8 fathoms, as stated by the author. There the action would take place at greater depth than on the Atlantic Coast, where the water is much more shallow and the foreshore much flatter.

The first problem that must be solved when jetties are to be used is how to guide the ebb currents so as to direct their force to and at the point where needed; in other words, to prevent the water of the ebb from spreading and losing its force before it reaches the bar. For that reason, in almost all cases, it is essential to hold the jetties close together to some point. In the case of converging jetties, the shore ends have effect on the direction of the current only through the sand impounded between them and the channel. When the channel near the shore is stable, this form of jetties is good, otherwise it is not. Where work is to be done in a bar, the head of the outflowing water must be held until the bar is reached, and for this it is evident that the jetties must be continuous from the shore out. Otherwise, there would be spreading of the channel flow close inshore, with a loss of effective force.

Much has been said about a jetty to the leeward and a jetty to the windward, and yet if it is considered that the head must be conserved until the point for work is reached, it is evident that one jetty will rarely do its work. Taking the sand movement on the South Atlantic Coast from north to south, if the jetty is placed on the north side the sand will be impounded for a certain time, and the currents, which in general leave the shore at right angles to the coast, will be continued in direction. But there is nothing to prevent the spreading before the working point is reached, which is frequently a mile from the coast.

If the jetty is placed on the south, concave to the current, in order to hold it, when the sand movement is sufficiently marked, making the bar obstinate, as it were, shallow false channels will branch out to the north inside the channel bar, and the effective force will again be lost. In one case the sand is kept away; in the other case the current is conserved a little better; but in both cases results are disappointing. Did time permit, these statements could be illustrated from the history of both the St. Johns River and Galveston improvements. Mr. Black.

Suppose the jetties in place and the channel scoured out. Nothing has been done to stop the sand movement. It makes no difference how high the jetty is built. In course of time, sufficient sand will be impounded to renew the sand movement across it and around its end. Building the jetty high with the hope of impounding all the sand that is going to move in the next fifty years seems rather futile, and does not appeal to the speaker.

It is believed that in all the harbors on the Atlantic Coast, and to a less extent on the Pacific Coast (because the movement there seems very much slower than on the Atlantic Coast), it is necessary first of all to have the jetties so placed as to cause the currents to bore through the bar, and to maintain the channel in a fixed position, so that it cannot shift under storm action; and then it is further necessary to take care of the constantly incoming sand exactly as has been necessary in the very much smaller jetties of the north of France. The amount of sand coming in year by year can be measured. At the mouth of the St. Johns it was measured quite accurately.

The jetties hold the channel. The annual accumulations must be removed by dredging, and if, in the work of cutting the channel originally, dredging be used, there will be little or no movement of the bar seaward. The dredging can be done between the jetties, or on the side from which the obstructing movement comes, as is frequently the custom in France, making a basin in which the sand or shingle will be deposited, and in that way not interfering with navigation. The channels in our country are ordinarily so broad that a dredge can be worked in them efficiently. In the French jetties, the channel is so narrow that a dredge cannot operate there without interfering with navigation.

There has been something said in this discussion about the comparison of maps. Anyone who knows these unstable bars very well knows that unless a map is made about once a month, map study may be very misleading, because the conditions change radically from month to month. Usually, on the Atlantic Coast northeast gales begin in September and last until April. In the summer time southeast gales are not rare. The map will show the results of the last storm action, not necessarily of the resultant storm action, nor the normal conditions. So it is dangerous to draw close conclusions from maps taken at long intervals.

Mr. Black. In regard to the use of dynamite at Brunswick, the speaker's deductions from the published reports are quite the reverse of those formed by Professor Haupt.

To the speaker, it seemed that the use of dynamite has been absolutely of no avail. The effects of the stirring up of sand by any means —by propeller dredging or anything else—is well known. There is no difficulty at all in getting any depth wanted by propeller dredging. The only difficulty is that the depth is lost in 20 or 30 ft., because it requires quite a high velocity and constant agitation to keep sand in motion. It must be stirred up and then kept moving. Propeller dredging has been resorted to very efficaciously in a narrow bar bounded by deep water, but over a broad bar, any local effect of that kind is bound to be local, and what is gained in one place is lost in another. The current will not carry sand for a long distance. Captain Gillette's paper, to which Professor Haupt alludes, is of great interest. Captain Gillette recommends the use of dredging in the Brunswick bar, believing that the channel will be sufficiently stable. Perhaps it will; perhaps it will not. In the absence of intimate knowledge of the local conditions, it is impossible for outsiders to judge.

The conditions on the Pacific Coast at Coos Bay seemed to the speaker, in reading the report and in looking at the maps, obscure. The resultant storms along that coast undoubtedly are from the south. The movement of sand at the mouth of the Columbia River is undoubtedly from the south, and yet the formation at Coos Bay would seem to show a local movement of sand from the north. That might come from some local current or wind action due to a bight in the coast. During the past year many theories have been advanced relating to the disposal of the sewage of the City of Havana. Everybody knows that the Gulf Stream flows between the Florida Reefs and the coast of Cuba from west to east, and flows with much velocity, and it is taken for granted that there is a current from west to east on the north coast of Cuba. As a matter of fact, the City of Havana is in a bight, and the current there is almost invariably, with very few exceptions in the month, decidedly from east to west. So that where there is a general movement along a coast in a certain direction, if there is a bight or local cause of that kind, an eddy is very apt to be formed, and the local movement may be in exactly the opposite direction. From looking at the maps, the speaker judges there must be some local sand movement at Coos Bay, a movement close inshore from north to south. These things are only mentioned in order to emphasize the fact that it is impossible to judge of the conditions at any point from maps taken at long intervals and without an intimate knowledge.

Mr. Ripley. H. C. RIPLEY, M. Am. Soc. C. E. (by letter).—The author gives an interesting description of the general physical characteristics of the North Pacific Coast and of the local conditions found at Coos Bay be-

fore work was commenced. He also gives a very fair diagnosis of the case and describes the treatment to be applied, and, so far as the work has been executed, he seems to regard the results as eminently successful, although he deems it probable that the south jetty will have to be built before the undesirable fluctuations in channel depth can be controlled. However, he apparently realizes the possibility of effecting an improvement by means of one jetty, where he says: Mr. Ripley.

"It was clearly evident that the north jetty would certainly be necessary, but, before commencing work, it was not so plain that the south jetty could not be dispensed with."

It will be interesting, therefore, if he will explain how the north jetty could be expected to intercept the sand drift from the south, which is the prevailing littoral movement, and why he should expect results at Coos Bay different from those at Charleston, S. C.; Cumberland Sound, Ga. and Fla.; St. Johns River, Fla.; Galveston and Aransas Pass, Texas, and many other places on the Atlantic and Gulf coasts where there have been expended many millions of dollars in experiments of this nature.

If the author will put in the 22-ft. contour of depth on the map of the August, 1899, survey (Fig. 5), he will find an area of deep water lying to the north of the north jetty having a length of over 2 000 ft. and a maximum depth of 37 ft. Had the south jetty been built first, this deep-water area would have been in the channel instead of behind the jetty, and, with the channel defended from the sand movement from the south, an increased channel depth of considerable degree of permanence would have been maintained. It seems, therefore, that the north jetty was certainly not the one to build first, and, if either jetty could be omitted entirely, it would be the north jetty that could be dispensed with.

If we examine a little more into the details of the changes, we find that from 1885 to 1899 the bar, as shown by the 18-ft. contour outside, has advanced 2 000 ft., and from 1894 to 1899 the channel width, between the 18-ft. contour and the jetty, has diminished from 800 ft. to 425 ft. and the south shoal has completely overlapped the entrance channel.

The author states that, with one exception, in November, 1899, when the depth shoaled for several weeks to 9 ft., due to a long and severe southwesterly storm, a depth of from 18 to 22 ft. has been maintained for seven years. It will be seen, however, by reference to the map of the 1899 survey, that a vessel, to avail itself of the best depth of water, would have to cross the bar broadside to the sea and make a right-angle turn near the outer end of the jetty before making the entrance —a feat quite impossible of accomplishment except for the shortest vessels and when the sea is comparatively smooth. But he, nevertheless, assures us that, thus far, this improvement has been eminently

Mr. Ripley.

successful. When it is remembered, however, that the most important function in determining the commercial value of a port is the uniformity of available depth, and that a fluctuation of as much as 13 ft. is possible here, it is not so apparent wherein this work has been so eminently successful.

That the work has produced important changes, there is no gainsaying. It is equally true that the average depth of entrance has been increased. But these changes have been largely subject to the caprice of wind and wave, and have in no sense been under the control of the works. The advance of the bar will increase the cost of a second jetty, and the complete accomplishment of a satisfactory improvement will be more expensive than had this work not been done at all.

The author, however, seems to realize the real essentials to success when he says:

"If works can be devised to prevent the encroachment of the sand on the channel and at the same time prevent the divergence and dissipation of the effect of the ebb tides, by confining and directing these tidal currents at a selected point of the bar, improvement may be confidently expected."

And it is equally apparent that the works thus far have not in any measure met the conditions here imposed.

If the author had read and appreciated the valuable writings of L. M. Haupt, M. Am. Soc. C. E., and others, on this subject, read before numerous scientific organizations of this country within the last fifteen years, the discussions before this Society by its members, the valuable papers read at the meetings of the International Navigation Congress, and if he had seen the results of a single jetty properly located, at the Malamocco entrance to the Harbor of Venice, and of the remarkable results from the reaction or curved breakwater, partially completed at Aransas Pass, Tex., he would have realized the possibility of fulfilling the foregoing conditions and thereby effecting a complete and permanent improvement by means of a single curved structure, either detached from or attached to the shore, and at a cost of from 25% to 50% of the cost under the adopted plan.

The writer's experience with brush mattresses agrees with that of the author. It was found on the Texas coast, as long ago as 1885, that stone made a much better protection from scour and undermining than brush work, and, where the teredo operates, it is much cheaper than brush, even when stone costs as much as $4 to $5 per ton.

Mr. McKinstry.

C. H. McKinstry, M. Am. Soc. C. E. (by letter).—The jetty at Coos Bay is a straight jetty built on the lee side of the entrance and connected with the shore. The jetty at Aransas Pass, to which reference has been made in two of the discussions, has a trace which is a "compound and reverse" curve, is built on the windward side of the entrance, and is detached from the shore.

The first is an example of the "single lee" jetty system; the second is a particular case of the "single windward" system. Mr. McKinstry.

The writer offers a few remarks on these two systems, with particular reference to conditions existing on the South Atlantic and Pacific Coasts of the United States.

A few preliminary remarks on littoral drift, and the effect of groynes and jetties upon it, may conduce to clearness.

*Littoral Drift.**—Surf raised by winds blowing along shore or obliquely on shore is the principal cause of littoral drift. The relative effect of this and of other causes is well brought out in the discussion following Mr. W. H. Wheeler's article,† in which the proposition is advanced that the principal agent in the transport of material along shores is the "wave-action of the flood tide." Strong littoral currents, pure and simple, and strong littoral tidal currents increase the transporting power of the surf when they coincide in direction with the wind producing the surf, and decrease it when they are opposed to that direction; but, in general, their sphere of action is not close in shore, but is in deep water.

There are no littoral currents, strictly so-called, along the South Atlantic Coast. Along the coast of Oregon and Washington there is a northward littoral current, an eddy from the Japan Current, but it is of slight velocity.

Along the coasts of Georgia and Florida there are littoral tidal currents, northerly during flood and southerly during ebb, due to the direction in which the tidal wave enters the great bay lying to the west of a line joining Hatteras and Canaveral. But these currents are of slight velocity, and are checked and even reversed by strong storm winds. The littoral tidal currents on the Pacific Coast are likewise of slight velocity.

In other words, the drift on these coasts, as on the coasts of tideless seas, is due almost wholly to surf raised by the wind.

The waves break and run in obliquely, while the receding water flows down the beach in a line nearly perpendicular to the shore. The effect of this zigzag motion of the breaking waves is a current which at times is of great velocity; strong enough often to sweep bathers off their feet. This is General Gillmore's "surf-current." Shield describes it as the "run, or set along shore, resulting from wave-action."‡ The drift of the water to leeward at the extreme inner edge of the broken waves is too well known to need remark. From the shore out to a depth depending on their size, the waves stir up material from the bottom and keep it in suspension, and it moves along

* The movement of sand by wind, above the water line, will not be touched on.

† "Littoral Drift: in its relation to the Outfalls of Rivers, and to the Construction and Maintenance of Harbours on Sandy Coasts."—*Minutes of Proceedings*, Institution of Civil Engineers, Vol. cxxv, 1895-96, p. 20.

‡ "Principles and Practice of Harbour Construction."

Mr. McKinstry. with the currents. On many coasts the discoloration of the water, due to this suspended material, is distinctly observable. Besides producing surf more or less heavy, a strong wind blowing along shore or obliquely on shore produces a littoral current, which, to distinguish it, may be called the "littoral storm current."

Within a wide zone, the movement of material is due to the littoral current; near shore it is caused by the surf current, while along the edge of the water it is caused directly by the broken waves. By the first two the material is carried in suspension; by the third, it is pushed bodily along. Doubtless, however, the surf current is often strong enough to roll material along the bottom as well as to carry it in suspension. In a cross-section at right angles to the shore the amount of material in motion varies from a maximum close in shore to zero at the limit of wave disturbance.*

On the South Atlantic Coast the drift is to the north in southerly winds, and to the south in northerly winds, but there is a strong resultant drift to the south due to the fact that the prevailing storm winds are from the northeast.

At Humboldt Bay, Cal., there is said to be no well-defined resultant drift, the movement to the south during the summer about counterbalancing that to the north during the winter.

At the mouth of the Columbia River the resultant drift is to the north.

Groynes and Jetties.—One effect of a groyne is to cause, during an alongshore storm, a strong current past the pier-head. A "cause of acceleration of tidal currents" is "that resulting from capes or headlands ponding back the water. At such points the main body of water to seaward is, by its inertia, slow to give way * * * to these irregularities of coast line, the result being that the passage or transfer of the inshore water from one side of the promontory to the other is obstructed."†

At Ymuiden, as Shield goes on to say, the building of the mile-long piers increased the current from its original velocity along that coast—2 to 2½ knots—to from 3 to 4 knots past the harbor mouth.

Our jetties produce the same effect upon the littoral storm current, as is made evident by the scouring of "pot holes" ahead of advancing jetties, and the resulting necessity of building long lengths of foundation ahead of the main work.

The existence of this accelerated current past the pier-head points to the advisability of curving the jetty off to leeward in order that the issuing harbor current may merge with this cross-current with a minimum of interference and disturbance.

Another effect, of building a groyne on a shore along which there

* The foregoing outline of the transporting action of the waves and of the "littoral storm current," is taken largely from *Engineering* for August, 1893.

† Shield.

is a movement of material, is an advance of the foreshore on its windward side. Whether there will also be a deposit in the lee angle depends on many circumstances, as, for example, on the strength of the winds that cause a littoral current and surf in the opposite direction to that of the prevailing drift, on the depth of water into which the groyne extends, and the velocity of the littoral storm current past the end. There has been an enormous deposit in the lee angle of the lee jetty at the St. Johns, and at other South Atlantic harbors. Mr. McKinstry.

It is believed by some that the most severe storms do not cause these deposits in the weather angle; but that at such times the deposits are cut away, the accompanying high tide and heavy surf producing powerful scouring undercurrents.

The building of a groyne or jetty does not, of course, stop the flow of the main portion of the littoral storm current and its suspended sand, but simply deflects it seaward. But it does intercept, for a time at least, the sand moved along by the waves and by the surf current. As a dam suddenly thrown across a stream would stop the flow of the water until the pool had risen to its crest, so a groyne built out into deep water stops for a time the flow of the river of sand traveling along the beach. Opinion is divided as to the extent to which the weather angle will fill up and the extent to which deposits will form on the ocean floor in the neighborhood of the pier-head. It has been held* that the "quantity of drift is limited," and that if the groyne be built into deep enough water, the drift "may be entirely stopped or its movement controlled." The author of these propositions considered that 4 fathoms was a sufficient depth, meaning, it must be, not that the drift would be permanently stopped at the groyne, but that after a certain amount of deposit had taken place in the weather angle the drift would be diverted into deep water and there dissipated. Experience at such places as Madras, Colombo, Malamocco, Newhaven and Ymuiden indicates that after a time the accretion in the weather angle and the advance of the foreshore become extremely slow. Cessation of the advance of the foreshore marks the beginning of the movement of drift past the groyne. What becomes of this drift? If it should be deposited over a comparatively small area in the neighborhood of the pier-head the growth of shoals in that neighborhood would be rapid. In certain of the cases mentioned the growth of such shoals is imperceptible and in the others very, very slow. From this it is plain that, under favorable circumstances, jetties, if long enough, will deflect the drift into deep water where it will be carried away by deep-water currents or else deposited over such a large area that the formation of shoals may be prevented by an amount of dredging not prohibitively great.

The time required to fill the weather angle depends upon the

* *Minutes of Proceedings*, Institution of Civil Engineers, Vol. cxxv, 1895-96, p. 2.

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quantity of sand moving along the coast and upon the length of the jetty and the depth into which it extends; curving the end of the jetty off to leeward increases the area over which the drift, if it be not carried away by deep-water currents, will be deposited; and an outflowing harbor current does much to deflect still further seaward the current of drift already deflected by the jetty, thus enlarging the area of deposit and tending to dissipate the drift.

At Port Said, when the drift began to deposit across the entrance, the engineers in charge removed sections of the upper part of the west breakwater to allow the drift to deposit in its lee; these deposits being removed from time to time by dredging. They preferred to deal with the drift in a place where dredges could work with but occasional interruption rather than in an outer bar. This expedient should be less necessary at an entrance having an extensive backwater and a considerable rise and fall of tide than at a place like Port Said. Gaps, or low places near shore, have been left in several weather jetties in the United States, but not for the purpose for which they were made at Port Said.

The Single Windward Jetty.—The earliest description of the concave windward jetty that the writer is aware of is contained in Mr. W. A. Brookes' treatise.* Says Wheeler, in "Tidal Rivers":

"Mr. Brookes also contended that in training a tidal river there is an advantage in giving a preponderating direction of the stream to one side of the channel by rendering it slightly concave. By this means he contends that a greater depth will be maintained in the channel at low water than in a straight channel; that, in training work carried out in sandy estuaries, a single concave training wall will be frequently found sufficient to maintain the channel in one course; and that piers at the mouths of rivers should always effect their junction with the sea by a concave pier on the windward side, by which means the outer division or convex side of the pier would shelter the interior of the channel from the prevailing gales, and prevent the range of the waves running along the face of the pier."

Elsewhere, in "Tidal Rivers," the theory of the action of the single windward jetty is thus explained:

"In determining the position to be given to training walls in estuaries where they discharge into the sea, the direction not only of the flood and ebb tides and of the prevailing winds must be considered, but also the amount of material that is transported along the coast, and the direction from which it comes. In some cases where there is much littoral drift, especially of shingle, the outfalls have been driven considerably to leeward. In such cases Nature appears to indicate that the water should be directed into the sea by a curved channel having its convex side presented to the direction from which the material is travelling, and that the training should be so designed that it may be extended further out seawards as the shore grows up with the accumulated material. * * * Where one stream joins another, the best direction for the junction of the tributary is by a gentle curve tangen-

* "Improvement of the Navigation of Tidal Rivers," 1841.

tial to the main stream. If this rule were applicable to tidal channels entering the sea, the outfall should be by a curved channel leading in the direction of the main set of the flood tide* along the coast. * * * It may be accepted as correct that, * * * the training walls of rivers, * * * to be successful, should never terminate in shallow water. * * * Where training walls have not been carried out into sufficiently deep water, their effect has been short-lived. Acting as a groyne to stop the travel of the littoral drift, this has rapidly accumulated at the back, and then found its way round the end into the mouth of the river, forming shoals or a bar. Where the pier reaches the level of deep water, the sand or shingle is kept in continual agitation by the effect of gales and the tidal currents, and is either drifted past the entrance or out into the deeper part of the sea. * * * There are cases where a single curved wall, running out from the shore line, has been sufficient to maintain a channel in the direction required. Such a wall placed on the windward side of the channel. * * * and presenting its convex side to the line of drift, if carried sufficiently far out into deep water, will give under favorable circumstances a permanent direction to the flow of water, and afford protection to the entrance from the prevailing gales. The jetty, acting as a groyne, will at first collect the littoral drift in the angle between the pier and the shore, but after this is filled up, the direction given by the convex form of the outer end of the jetty will carry the drift beyond the entrance and into the deep-water currents. The flood tide," (read "littoral storm current") "working round the end of the pier-head into the channel, as it does in the case of the natural pier at Spurn Point in the Humber, will tend to maintain at that point deep water, and prevent the deposit of the littoral drift and the formation of a bar. The ebb current running along the concave side of the jetty will maintain the deepest water along that side of the channel, the flood tide setting up also along the line of deepest water. A uniform and deep channel may thus be maintained. If the line of direction be well chosen with regard to existing circumstances, a single wall will therefore be sufficient to maintain a channel in a uniform direction and preserve deep water. There are several examples where single walls thus carried out have been successful."

Mr. McKinstry.

Certain of the matters brought out in this extract deserve more particular mention:

1.—"A single windward jetty will often be sufficient to fix an entrance channel."

This is the contention of Professor Haupt, with respect to the reaction breakwater.

He states† that the Aransas Pass jetty was "so adjusted, with reference to the axis of discharge, as to receive the same tangentially," but, as is clearly pointed out by Captain Gillette, in the Report on Brunswick Bar,‡ the axis of discharge owes its position to the Nelson jetty. When work began at Charleston the main channel was parallel to the coast, and the bar at its outer end was 5 miles from the harbor entrance. Let one attempt to adjust a reaction breakwater to these conditions. If

* To adapt this to conditions at United States harbors, for "flood tide" read "littoral storm current."

† *Transactions*, Am. Soc. C. E., Vol. xlii, p. 492.

‡ House Ex. Doc., No. 355, 56th Cong., 2d Sess.

Mr. McKinstry.

placed on the bar and adjusted to the axis of the main channel, it would have to be some 5 or 6 miles long to prevent the channel from breaking out to northward of it. If adjusted to the axis of one of the minor channels, only a small portion of the ebb and flow would be controlled. Similarly, with a Brookes jetty. The necessity of a lee directing jetty in connection with the concave windward (Brookes) jetty, proposed by Professor Haupt for the improvement of the Southwest Pass, is also plainly shown in Captain Gillette's report.

In fact, in order that the concave jetty may do the work expected of it, there must be something on the lee side of the entrance, either a natural bank of the proper shape, or else a jetty, to direct the current into the concavity of the windward jetty.

Accordingly, in existing cases, these supplementary jetties are found to vary from short jetties, like the Nelson jetty at Aransas Pass and the Arenas mole at Bilbao, to jetties perhaps half as long as the main structure, like the south jetty at the Malamocco entrance.

Swinemunde is an example of a complete reversal of this arrangement, for there the longer or main jetty is on the lee side of the entrance, and the windward jetty is convex instead of concave to the channel.

Strictly speaking, the Aransas Pass improvement is not a single-jetty improvement, in spite of the claims of its designers. It is inferable from Mr. Ripley's words* that he regards the Malamocco entrance as such an improvement. He cannot, of course, be unaware of the existence of the south jetty at that place; he must mean to deny to it all share in the improvement effected.

2.—It is advantageous to present a convex curve to the direction of drift. At the same time, a concave bank is thus afforded to the currents in the entrance channel.

A notable departure from these principles is the recently proposed extension, concave to windward, of the jetty at the mouth of the Columbia River. There is a local reason why the existing jetty at that place was built concave.

3.—It is advantageous to direct the ebb current into the littoral current tangentially or at a small angle. This was referred to under the heading "Groynes and Jetties."

4.—It is absolutely necessary to carry the jetty out to a depth in which the waves are powerless to stir up the bottom.

5.—After the weather angle has filled up, "the direction given by the convex form of the outer end of the jetty will carry the drift beyond the entrance and into the deep-water currents." This, also, was referred to under the heading "Groynes and Jetties."

It may be interesting to investigate to what extent this opinion is shared by other writers and borne out by experience.

Of Malamocco, Vernon-Harcourt says:

* "If he had seen the results of a single jetty properly located, at the Malamocco entrance to the Harbor of Venice. . . ." p. 532.

"The checking of the littoral current has reduced the depths on the northern side; but this diminution does not extend beyond depths of between 16 and 20 ft., so it has not hitherto affected the pass" Mr. McKinstry.

He also says:

"It is true that at all these places (Sunderland, Ymuiden, Dublin, Charleston), an advance of the shore has occurred; but its progress seems to have ceased at Ymuiden and Sunderland, it is very gradual at Dublin, and will probably prove equally so in other instances. Even at Port Said, which it was feared might be imperilled by the alluvial drift from the delta of the Nile, and which is undoubtedly exposed to a turbid current from the west, the advance of the western foreshore has become very slight."

The following is taken from Shield's "Principles and Practice of Harbor Construction," page 285:

"They (the waves) also heap up drift materials against obstacles, such as piers, etc., which run across the shore along which the drift travels; but when such piers extend into deep water, a considerable time often elapses before the drift material—as distinguished from that held in suspension—finds its way into the harbor, because the corners which the piers form with the coast constitute traps for it. As, however, these fill up, an advance of the foreshore takes place, and the material finds its way out to and round the pier-heads. Should these occupy a salient position, and extend well into the tide-way (for our harbors, read 'littoral storm current') the current past them will scour much of this away, and enable the depth of water at the entrance to be more easily maintained than under other conditions. Nevertheless, a current in passing an entrance often expands into the harbor and creates eddies, by means of which some of the drift material is almost sure to be carried in, and deposited under the lee of the piers. Total immunity from dredging can therefore scarcely be expected.

"In order to combat the advance of foreshores, jetties are often extended seaward; but such prolongations, unaided by dredging, are generally found to be of little permanent benefit, inasmuch as each extension of the pier causes a corresponding advance of the shore. The rate of such advance depends upon the physical conditions peculiar to each site, and especially upon the quantity of sand in motion. This varies greatly in different localities—hence, while the extensions of the jetties at such ports as Calais and Dunkirk have failed to produce the desired results, the prolongation of those at Port Said has been more successful; so much so, indeed, that it has been estimated it will take three centuries for the foreshore at Port Said to advance to the end of the piers, assuming its present rate of accumulation to be maintained.

"In order to keep open the Ymuiden harbor, it has been found necessary to dredge, on an average, about 650 000 cu. yds. of sand per annum, at an average cost of about £22 000.

"Such an expenditure as this can only be met by a very large traffic, and there can be no doubt that, but for this, Ymuiden harbor would ere now have ranked amongst the failures of harbors on sandy coasts."*

Wheeler illustrates his opinion by the following, relating to Lowestoft:

"Where it is contemplated to run out jetties into the sea at the

* This deposit is of mud, and is brought into the harbor by the flood tide, and not by an advance of the foreshore around the jetty ends. There is no outer bar.

Mr McKinstry. mouth of a tidal river in order to remove a shoal or bar, the risk of the gradual growth of the shore-line seawards until it progresses as far out as the pier has to be considered. Thus for the protection of the harbour of Lowestoft, situated on the flat shore of the East Coast, piers were built out from the shore-line. The windward pier, or that on the side from which the drift came, acted as a groyne, causing the sand and shingle to accumulate at the back and to extend a considerable distance out from the shore. * * * The shingle is constantly working its way round the end of the north pier into the harbour, necessitating constant dredging to maintain the depth. In north-east gales a bar also forms across the entrance, interfering with the navigation. In this case the jetty does not extend sufficiently far out to prevent the shingle travelling round the end, or to make the velocity of the littoral current sufficient to carry it past the pier-heads. The accumulation of shingle having now extended out from the shore past the piers, there is always at hand a constant supply of material for the littoral current to carry into the harbour. This could be prevented by arresting the travel of the shingle by a groyne carried out from the Ness, which projects from the coast-line about three-quarters of a mile to the north of the harbour entrance, and which could be extended from time to time as the material accumulated at the back."

To sum up: A windward jetty, especially a concave windward jetty, may be expected not to cause a dangerous advance of the foreshore when conditions are favorable, *i. e.*, when the entrance is not in the bight of a shallow bay, when the jetty is built out into deep water and its end is swept by a strong cross-current.

For, "in such a case," to apply to our case what Vernon-Harcourt says of converging-breakwater harbors, "sand does not come in through the entrance, as waves do not stir up sand in deep water; the current round the extremity assists in maintaining the depth; and owing to the depth which the advancing foreshore has to fill up and the projection from the shore which it has to assume, it may take a long period in reaching the end of the breakwater opposed to the littoral drift, or previously attain a position of equilibrium."

In applying this or any other system to any given place, the possibility would naturally be kept in mind that a bar might form beyond the entrance; and the location of the jetty and the curve given to its outer end would be such as not to preclude its extension. On the other hand, modern dredging methods afford a less expensive means of maintaining entrance channels than do periodical jetty extensions. Moreover, as DeMay says, the actual removal by dredges of the deposits which form before entrances has shown them to be of much less volume than was formerly thought.

For example, to maintain, not to speak of producing, a 21-ft. channel at Charleston, would require the inner ends of both jetties to be built up to about high-water level, and both to be extended out to, and probably beyond, the crest of the bar—not the bar of to-day, but the bar in any more remote position that it might assume as a result of this jetty work. Even this might fail to produce the desired result. The interest on the cost of this work would be greater than the sum it

is now costing to maintain the desired channel by dredging. By reason of these recent improvements in dredging, dredging alone may afford a cheaper method of improvement than jetty work alone or jetty work supplemented by dredging. This, for example, was the conclusion recently arrived at for Brunswick Bar. By Mr. McKinstry.

In beginning work under any given system of jetty improvement, a channel of something like the required depth and width may or may not lie in the position selected for the final channel. If it does not, the amount of material to be moved to form the desired channel may be very large, and if it moves slowly seaward under the influence of the advancing jetties, it may form a bar. When once the jetties have been completed and the desired channel has been produced either by scour or dredging, it is expected that sand deposited in the channel by a combination of unfavorable circumstances will be picked up, on the return of normal circumstances, carried to sea and dispersed. Circumstances, however, may defeat this expectation, and make dredging necessary. But while the jetties are in process of construction, the effect of steadily adding to their height and of steadily lessening the distance between them, is to increase the current between them to one that is just strong enough to scour. (This assumes that the required channel is not formed by dredging, and that the flow through other channels is more or less throttled by the jetties.) The transporting power of the current between the jetties is, of course, very small, and its load will be deposited upon the slightest diminution of velocity, that is, will be deposited just beyond the jetty ends, wherever they happen to be at the moment. The ultimate result is that much of the sand that occupied the locus of the desired channel has been shifted from its original position to one just beyond the jetty ends, in their final position. The tendency for the material to deposit in prolongation of the channel will decrease as the jetties are extended, for the current past the jetty ends will increase as the jetties grow longer, and will deflect the sand-laden issuing current to leeward. The above possible cause of the formation of bars is independent of the system of jetty improvement adopted, and indicates that in many cases it may be advisable to dredge while the jetties are advancing. There is no doubt in the writer's mind that the present bars at Charleston and at the St. Johns were largely formed in this way, for at both places the quantity of sand scoured out to form the present channels was enormous, and the depth beyond the jetty ends comparatively slight. Doubtless, however, in both cases, considerable of the sand in the present bars came in over the low places in the weather jetties.

To digress a moment, one cannot fail to notice, in the strictures passed on the Charleston improvement, a lack of fairness, or at least an absence of broad-minded consideration of the peculiarly unfavorable conditions surrounding that work. The place possesses about

Mr. McKinstry. every drawback that can be mentioned: It is situated in the bight of a bay, the rise and fall of the tide is not great, the quantity of sand moving along the coast is enormous, the water off the entrance is shoal, and the slope of the ocean floor is very flat. The improvement was begun more than 25 years ago, the yearly appropriations for it have been less than 5% of the total estimated cost, and time enough has elapsed to show the effect of the work done. What a magnificent field for a display of "hindsight"!

Compare the conditions as to depth at Coos Bay and at Charleston. At the former place, when the improvement was begun, there was a depth of 35 ft. at the distance of 1 mile from the harbor throat; at Charleston this depth was found 8 miles off the entrance. Again, where the Coos Bay jetty now ends there was a depth of 31 ft., and the channel depth to be obtained was 20 ft.; at Charleston, where the jetties now end there was a depth of only 18 ft., while 21 ft. was the channel depth desired. The above-mentioned unfavorable conditions; leaving a long, low place in the weather jetty close to its inner end, and not resorting earlier to dredging, are the causes of the present outer bar at Charleston—a bar which the current issuing from between the present jetties is powerless to scour. The success of the improvement is due to the work of sea-going suction dredges.

The "reaction breakwater" of Professor Haupt is a particular case of the single windward jetty, being a high windward jetty, detached from the shore and having a trace which is a "compound and reverse" curve.

Professor Haupt wishes a clear distinction made between the reaction breakwater and any single jetty rooted to the shore; for he says*:

"Again, it is attempted to compare a single jetty projecting from the shore with the reaction breakwater which is purposely wholly detached therefrom."

Detachment from the shore, and the fact that the inner portion of the reaction breakwater is convex to the channel, are the two points of difference between that structure and the Brookes concave windward jetty. But the inner ends of many weather jetties in this country are convex to the channel: Witness, Charleston, Cumberland Sound and the St. Johns; and many of our jetties are detached jetties, or, what is the same thing, have the crests of their shore ends below low water—often well below that level.

The expressed object of placing the reaction breakwater to windward of the entrance is "to arrest the littoral drift"; but to leave a gap is to fail to arrest the drift at and near the shore where the amount of drift is greatest. At Port Said, as above mentioned, provision is made to remove, by dredging, the drift that comes in through the openings made in the weather breakwater; but dredging forms no part of the reaction breakwater system. The object of the gap is "to admit the

* *Transactions*, Am. Soc. C. E., Vol. xlii, p. 530.

full tidal prism." General Gillmore states that in a certain "flood" channel at Cumberland Sound—a channel close to the beach on the north side of the entrance—the gauged flow on the flood was about double that on the ebb. Leaving gaps is undoubtedly a means of increasing the preponderance of the ebb over the flood in the main channel, provided this channel is not too long. At Cumberland Sound and at the St. Johns this effect is assisted by the fact that the ebb current does not begin to run until after the tide has fallen about one-quarter of its range, and continues running until the water has risen about the same amount. But it may fairly be doubted whether the advantage accruing from a long, deep gap would offset the evil effect of offering a capacious opening to the drift, or whether a gap that would keep out drift except at highest water would materially increase the ebb's preponderance. Attempts have been made, as at the Adour and at Dunkirk, to solve the problem by "movable" jetties, that is, jetties which can be opened during flood tide (not, however, during storms), and closed during the ebb. Mr McKinstry.

The object of the reaction breakwater's "compound and reverse curve" trace is stated to be to "produce reactions similar to those found in the concavities of streams." But this reason evidently cannot apply to the convex inner part of that structure. There is an excellent reason, to the writer's mind, why the inner part of the Aransas Pass jetty should be convex to the channel: In military parlance, it was necessary to refuse this flank to prevent its being turned. If it had not been given this shape there would have been little to prevent the channel from breaking out to the north of the jetty during a southwest storm. Similarly, half a dozen harbors can be named, at which, if an attempt should be made to lay down a Brookes' concave windward jetty, the main portion of the jetty would have to be connected with the shore by means of a convex portion, or a portion making an angle, measured on the channel side, of more than 180° with the main portion. The reason of this is that the weather shore of the entrance is usually farther from the axis of the channel than the jetty should be. Coos Bay might be an exception. The Southwest Pass is another exception, and Professor Haupt's plan for this place omits the convex curve. While there are no novel elements in the reaction breakwater, the combination of these elements is unique, and the reaction breakwater at Aransas Pass is the first concave windward jetty built at a tidal entrance in this country.*

The foregoing discussion indicates, generally, that when it is proposed to rely upon a windward jetty as the main means of improvement at a place where the resultant drift is considerable:

1.—The outer end of the jetty should be concave to the channel.

* This is taking for granted that the east side of the entrance is the windward side, which is disputed. If the west side is the windward side, then the original Government jetty was the first concave windward jetty built in this country.

Mr. McKinstry.

2.—The jetty should be connected with the shore, or, if a gap be left, provision must be made for dredging. In the general case it will be found necessary to make this connection by means of a curve convex to the channel.

3.—The jetty should be built up at least to high-water level, and should extend across the bar into deep water.

4.—In all cases except where the lee side of the entrance happens to be of the proper shape to direct the current into the concavity of the main jetty, a lee directing jetty, shorter than the main jetty, will be required. Where there is a strong resultant drift, the lee jetty need not be raised above the level of the beginning of ebb outflow.

The Single Lee Jetty.—Single lee jetties have been built straight, as at Coos Bay,* or convex to the channel, as at Grays Harbor.

All that has been said herein as to the advantage of directing the issuing ebb current into the littoral current with a minimum of disturbance, applies with equal force to the lee-jetty system. It is accomplished by making the jetty incline to leeward, or by making its outer end convex to the channel. As between a straight and a convex lee jetty, the advantage seems to lie with the latter.

The theory of the single lee jetty is this: Due to the prevailing storm winds and to the deposit of sand on the windward side of the issuing harbor current,† the channel is forced steadily to the leeward until it runs nearly parallel to the coast. Finally, its length and the friction of its bed become so great that on some favorable occasion, say during a strong off-shore blow following an on-shore storm and extreme high water, it breaks straight out through the bar, and its slow movement begins anew. A jetty built to leeward of the channel when the latter occupies, or lies to windward of, some favorable position, will intercept this steady movement to leeward; and, the pressure to leeward still continuing, the inflowing and outflowing waters confined against the jetty will scour a channel of the required depth in the selected location. The theory of the single lee jetty is set forth in various papers and discussions contributed to the *Transactions* of this Society by Major Symons.

The contention of the opponents of this system is that it tends to cause the formation of a bar about the entrance, beyond the jetty end. It may be interesting to inquire into this matter. As before stated, a jetty built out on a sandy coast invariably causes a deposit of sand on its windward side. When the jetty is located on the lee side of an entrance, the tendency to fill on its windward side,

* The jetty at Coos Bay is on the lee side of the entrance, considering the direction of the prevailing storm winds. Whether it is on the opposite side to that from which the maximum drift comes is not so certain. It is the opinion of one who has long been connected with Pacific Coast works that the resultant drift at this place is from north to south, on account of the fact that the irregularities in the shore line to the south interfere with the movement of sand from that direction, while there is nothing to obstruct the sand movement from the north.

† This subject is well discussed and illustrated in Shield's "Principles and Practice of Harbor Construction," page 151.

i. e., in the entrance, still exists, but is interfered with and prevented by the flow of the waters that make in and out of the harbor. What becomes of the sand that otherwise would be deposited in the windward angle of the jetty? Mr. McKinstry.

During storms from the prevailing direction, and while the flood current runs, the drift will be carried into the channel and the inner harbor; when the tide turns, the issuing current will deflect the drift-laden littoral current out to the end of the jetty. There the resistance to leeward movement ceases, and the united currents bend to leeward around the jetty end. All or nearly all of the sand that is carried into the inner harbor during the flood tide must be carried out again by the ebb current, otherwise the harbor would fill up. The issuing harbor current is therefore sand-laden, and joins its drift to that of the deflected littoral current. If the jetty is made long enough, this material will probably be dissipated in deep water.

The effect is the same in kind, if not in degree, when a low or detached weather jetty is used in connection with a high lee jetty. The efficacy of the weather jetty, in keeping the drift out of the channel and causing it to deposit on its windward side, depends upon the height and continuity of the jetty.

The tendency for a deposit to form along the windward side of the issuing ebb current, previously referred to, must be considered further. The same forces are at work here that act to cause an advance of the foreshore of the windward side of a groyne. During the flood the deposits on the windward side of the channel would suffer erosion, the eroded material being carried toward the inner harbor. The building up of shoals in the apex of the angle between the channel and the shore and along the windward side of the channel ought, therefore, to be slower than the corresponding deposit in the windward angle of a groyne or of a windward jetty, and such shoals would probably never rise to low-water level. Furthermore analogy indicates that if the jetty be made long enough and be carried into deep enough water the deposit ought not to overlap the entrance.

The formation of the south shoal at Coos Bay, however, looks like an instance of this overlapping. This shoal extends along the weather side of the channel, and at the outer end bends around parallel to what is probably the direction of the ebb current when deflected by the littoral storm current from the south. At a point in prolongation of the jetty, and about 1 200 ft. beyond its end, this shoal carries less than 18 ft. of water, where in 1885 there were more than 40 ft. A similar shoal formed in prolongation of the north jetty at the St. Johns, at one stage of the work, viz., after the south jetty had been built out considerably beyond the north. The overlapping shoal at Coos Bay is referred, by one commentator, to the jetty being on what he considers the wrong side of the entrance; but it may be due to the jetty not

Mr. McKinstry.

having been built out far enough. It is by no means certain that a windward jetty, built out no further, would not produce a shoal equally bad.

In stating that the single windward jetty should be concave to the channel (and hence convex to windward), and that the single lee jetty should be convex to the channel (and hence convex to windward) convex and concave to windward refer to the prevailing storm winds; and a preponderance of drift from that direction is assumed. When, under the influence of winds from certain other directions, littoral current and drift are in the direction opposite to the prevailing direction, certain of the advantages of the concave windward jetty are exchanged for disadvantages; thus, the ebb current is discharged into the littoral current up stream instead of down stream, an easy passage of the deflected littoral current around the jetty end is prevented, and the disturbance caused by the meeting of the currents is increased. Because, however, it is impossible for a single jetty to suit these opposite conditions, it is best to give it such a shape as best suits the storm conditions, especially if the preponderance of current and drift from the storm direction is considerable. Similarly, in the case of the single lee jetty, when littoral current and drift are opposed to the prevailing direction the "pressure to leeward" does not exist and there is nothing except the shape of the banks to hold the inflowing and outflowing currents to the channel. Here again, however, the storm conditions, being the more powerful, are the conditions to be accommodated.

At places where there is no preponderance of drift, the conditions must be met by two converging jetties, extending out to the same contour of depth, each part making a large obtuse angle with the shore.

In any case it should be noted that the trending of the ebb current to leeward under the influence of the littoral current flowing across the entrance—inducing a dissipation of material or a deposit elsewhere than in prolongation of the channel—may be prevented by a sudden change of wind.

From the point of view of ease and safety of navigation, the windward jetty is superior to the lee. The former stills the sea at the entrance—a matter of importance, because, in a heavy sea, the available depth is decreased by the drop of the waves as well as by the resulting "send" of the vessel. Thus, at the mouth of the Columbia River, a depth of 40 ft. is considered advisable for vessels that require but 30 ft. in still water.

Similarly, in the case of two jetties, an overlapping weather jetty will give some protection to entering vessels before they come abreast of the end of the lee jetty.

A single lee jetty is a "lee shore," and makes a dangerous sea. Note, again, however, that the terms "windward jetty" and "lee

jetty," as here used, apply only during storms from the prevailing direction. The designations change under certain changes of wind. Mr. McKinstry.

Physical and nautical requirements are in opposition in the case of entrances formed by the two sorts of jetties under consideration, and neither jetty gives an entrance facing in the most favorable direction for sailing vessels that may try to make the harbor during a storm from the prevailing direction. But this is not so important in these days when sailing vessels have been largely superseded by steam, and when coasting sailing vessels receive timely warning of the approach of storms.

The writer expresses the opinion—necessarily, under the circumstances, a merely academic opinion—that the concave windward jetty, supplemented by a lee jetty sufficiently long to direct the ebb current properly, is preferable to the single lee jetty.

In conclusion, a word on the text: "The sea seems almost to take a delight in thwarting a timorous or vacillating policy.* A well-thought-out plan having once been adopted, vigor and steadfastness should characterize its execution. Often, to be sure, irregular and insufficient appropriations, and the impossibility of doing anything that will temporarily decrease an existing navigable depth; no matter how beneficial the ultimate result may be, have prevented such a course on many of the works in this country. To build a windward jetty to low water in the hope that it may not be necessary to build it to high water; to leave a gap in a windward jetty for fear the tidal prism may be reduced, when, if necessary, dredging should be resorted to, to prevent this reduction; to build a jetty out to a certain point and then await results in the hope that a shorter jetty than the one originally planned may accomplish the end desired: These are methods of inviting an advance of the bar and greatly increased final expense.

William W. Harts, M. Am. Soc. C. E. (by letter).—The writer is well pleased with the discussion aroused by this paper, and the attention to the subject that has been apparently drawn by it. Mr. Harts.

Professor Haupt's discussion seems mainly to show a careful endeavor to explain away definite results achieved, and to defend the peculiar theory of erosion by "impact" or "reaction."

Stevenson's formula, referred to as showing the relation between wave height and fetch, was inserted in the paper merely to show that such a relationship was well recognized, but the formula was purposely not applied specifically to this locality.

There seems to be some confusion in Professor Haupt's use of the terms "windward" and "leeward" sides of the channel. It is possible that the sides of the channel from which come the worst and longest storms, and that from which the resultant sand movement comes, may not be the same. This appears to be the case at Coos Bay. It is not

* *Transactions*, Am. Soc. C. E., Vol. xxx, p. 499.

Mr. Harts. plain in such cases which side would answer the "rule of thumb" of "one jetty to the windward." Attention might properly be called to the danger of the indiscriminate application of rules of this character to cases which may and usually do differ as much as the various ocean ports of the United States, unless a careful study of the natural conditions of the place considered is also made.

The fact of the unusual depths of 30 ft. or more at mean lower low water, found across the bar at Coos Bay before improvement, during the occasional and short periods when the channel in its cyclic change of position went westward straight to sea, shows the possibilities of improvement at this place. In approximating to these possibilities, may not a practically permanent increase in the usual or ruling depth of more than double what it was originally, thus satisfying both the terms of the project and the present requirements of navigation, be termed a distinct success?

Also, it is not admitted by all engineers that there is any positive evidence in fact for the theory of erosion by "impact" of water particles in running streams. The well-known theory of the relation between bottom velocity and erosive and transporting powers still has its advocates.

Professor Haupt refers to the Second Board of Engineers, called to report upon the further improvement of Yaquina Bay, just after the completion of the first project. Attention might also have been called to the fact that, the first project having been satisfactorily completed, this Board was presumably called, more for political reasons than otherwise, to advise the Secretary of War as to the feasibility of methods of still further increasing the depths already secured.

In 1900, still another Board recommended that no further jetty work be done, depths being ample for all present needs. There has been for years an available channel of seldom less than 15 ft. depth at mean lower low water, and frequently of something more. The survey of 1900 was made after a period of very severe southwest storms, and although the best depths at the bar at that time, stated by Professor Haupt to be 8 or 10 ft., were actually 13 ft., it may be added that after a short period of smooth weather, this depth was again increased to the usual 15 ft., and about a month later to 16 ft.

It is the writer's belief that no form of improvement will prevent the occasional heaping up of sand in the channels of improved harbors on the Pacific Coast during severe storms, but that, with works properly designed and maintained, the concentrated current will invariably cause the channel to resume its usual depth when quiet weather prevails. It seems impossible to fix absolutely, on this coast, the balance between the shoaling effect of the sea in storms, on the one hand, and the scouring effect of the tidal flow, on the other.

Professor Haupt's reference to the improvement of the bar of the

harbor at Brunswick, Ga., by dynamite, seems particularly unfortunate, in view of the recent information* published by the War Department, concerning this port. Mr. Harts.

In this comprehensive and convincing report, made by Captain Gillette, Corps of Engineers, U. S. A., on the improvement of this harbor, the following appears in reference to the cost of ordinary dredging, etc.:

"This table shows the remarkable fact that up to 1897 (six years) the total result of work in the vicinity of the Goodyear channel was a fill instead of a scour. This was the period in which dynamite alone was used to do the work, aided only by harrows, rakes, and a dredge without bins, but not by ordinary hydraulic dredging. * * * The results for the total period from 1891 to 1898 are also quite meager—a maximum of 218 260 cu. yds., considering a width of 800 ft., 400 ft. each side of the axis of the present channel. For this there has been paid Mr. Goodyear $220 000, and the numerous surveys to determine the results of his work have cost $26 989.83, making a total of $246 989.83, or at the rate of $1.13 per cubic yard, assuming that the removal over this wide area was all useful, and that it was all done by Mr. Goodyear. * * * Fifteen cents per cubic yard is a fair price for this work. The contractor now working at Doboy Bar, the next bar north of Brunswick, and apparently of the same nature, gets 13.9 cents per yard, and he has had to build his dredge for this job. The price of 15 cents per yard may be objected to on the ground of 'tough blue clay' and 'sand, shells, and gravel cemented together.' The report of the original survey of 1891 states 'sand and blue mud,' the latter when under water being 'soft and very easily eroded.' It is to be noted, too, that Mr. Bryan, the dredging contractor who did the dredging for Mr. Goodyear, while arguing that 15 cents is too small, gives the reasons of 'distance from land' and the 'weather conditions,' but says not a word about tough materials (p. 54). The other two conditions are about the same at Doboy as at Brunswick. At 15 cents the entire work would have cost $18 750, a little over two-thirds the amount it has cost the Government for making the surveys to see what Mr. Goodyear had accomplished."

The conclusions drawn in regard to deepening by dynamite at this place are as follows:

"*First.*—Dynamite does not have any useful effect in bar deepening, except in loosening tough materials in the immediate vicinity of explosions. Such materials are very rare on bars along sandy coasts.

"*Second.*—The channel now existing at Brunswick Bar has been produced by nature, aided very materially by ordinary dredging. The useful effect of dynamite has been very small, except, possibly, to loosen materials in some places as an aid to the dredge.

"*Third.*—The cost to the Government of all material removed, whether usefully removed or otherwise, has been $1.13 a yard, more than seven times the cost of ordinary dredging. The cost to the contractor is said to have been even more than this.

"*Fourth.*—The channel produced has no advantage of permanence over a dredged channel of similar size and location, which could be produced for about one-twelfth of the cost of the present channel."

* H. Doc. 355, 56th Cong., 2d Sess.

Mr. Harts. Major Black's experience and views corroborate those of the writer, with regard to the use of two jetties in similar harbor improvement, and that the effect of a single jetty is but partial. He is believed to be right in his presumption, that the resultant sand movement, at this point, is from the north, in spite of the fact that the worst and longest storms come from the south and southwest.

Mr. Ripley, in his discussion, fails to grasp the fact before stated, that the resultant sand movement and the worst weather apparently come from different directions.

Would it not be evidently inadvisable, in such a case as this, to endeavor to control sand movement from the north by a jetty on the south ?

It is believed that Mr. Ripley's understanding of the description of this work must be considerably in error, for his reference to a "fluctuation of as much as 13 ft." intimates that no reliance by mariners is to be placed on the depths secured. The writer regrets exceedingly if his description conveys such a misapprehension to unprejudiced minds.

As to whether the results achieved at this place were originally produced and are still controlled by the jetty, or whether they are at the caprice of the wind and wave, may be safely left to the unbiased judgment of the reader, as well as the question of how well this work answers the conditions to be filled, before improvement may be expected.

The writer has visited the harbor of Venice, and has studied the effect of the Malamocco jetty work. He has also read a number of the valuable writings of Professor Haupt, and is still of the opinion that if the south jetty at Coos Bay had been built first, no valuable result would have been secured in concentrating the tidal flow over the bar, and in controlling the serious sand movement on the north side of the entrance; and that such work, if built as recommended by Professor Haupt and Mr. Ripley, would be practically valueless for the benefit of navigation, until a north jetty could be built to fulfill the functions now performed by the north jetty. This would have delayed for a number of years the valuable results already achieved.

MEMOIRS OF DECEASED MEMBERS.

HORATIO GOUVERNEUR WRIGHT, Hon. M. Am. Soc. C. E.*

DIED JULY 2D, 1899.

Horatio Gouverneur Wright was born at Clinton, Connecticut, on March 6th, 1820. He was seventh in lineal descent from Benjamin Wright, who was an officer under Charles I of England, and came over with other royalists to this country during the protectorate of Cromwell. He was also descended from the well-known New England families of Griswold, Chapman, Grinnell, Peabody and Alden.

He entered the United States Military Academy in 1837, and was graduated in 1841 with high honors. He was promoted in the army to Second-Lieutenant in the Corps of Engineers on July 1st, 1841, and remained in the Corps from that time until his retirement from active service on March 6th, 1884, a period of forty-seven years, passing through all the grades, including that of Brigadier-General and Chief of Engineers.

Soon after his graduation, he was assigned to duty as Assistant Professor at West Point, in which capacity he served for several years. He then served for about two years as assistant to the Board of Engineers having in charge the designing of the sea-coast defences of the country.

In 1846, General Wright commenced active practice as a constructing engineer, and he continued in this duty for about ten years. At this time the eminent military engineer, General Totten, was Chief of the Corps of Engineers, and under his direction the military defences of the country were being rapidly constructed upon a system which then had no superior in the world. The armaments and magazines were covered by masonry walls which were impenetrable to the naval fire of that day. Under General Totten's direction, the young engineer officers were required to make the detailed drawings for these works, to superintend the cutting of the stone, the mixing of the mortar, the laying-out of the work and the actual construction upon the ground. It was in this school that General Wright acquired that thorough practical knowledge of the details of construction which was so often observed by officers serving under him in later years. His principal service on this class of duty was in the construction of Fort Jefferson, at Tortugas, Florida, the largest fort ever built in this country, and Fort Taylor, at Key West, Florida.

* Memoir prepared by G. J. Fiebeger, J. L. Lusk and Charles Walker Raymond, Members, Am. Soc. C. E.

In 1856, General Wright was ordered to Washington as assistant to the Chief of the Corps, and he continued to occupy this responsible and arduous position until the outbreak of the Civil War in 1861.

The full record of General Wright's distinguished services as a military commander during the war cannot with propriety be given in this brief memoir, which relates rather to his history as an engineer than to his career as a soldier; but no account of his life would be satisfactory which omitted mention of his eminent military services during the Civil War. At the first battle of Bull Run he was Chief Engineer of Heintzelman's Division. He was commissioned a Brigadier-General of Volunteers on September 14th, 1861, and commanded a brigade of land forces of the Port Royal expedition. In February, 1862, he left Port Royal with a brigade of Volunteers, and by the middle of March had captured Fernandina, Jacksonville and St. Augustine, and retaken possession of Fort Marion and Fort Clinch. He was promoted Major-General of Volunteers in July, 1862, and commanded the Department of the Ohio until March, 1863. He led a division in the battle of Gettysburg and in the spirited assault on Rappahannock Station (November 7th, 1863). At Spottsylvania, on May 9th, 1864, he succeeded to the command of the Sixth Corps. He was present with the Army of the Potomac at every engagement up to July, 1864, when he was summoned from the front of Petersburg with his corps to the defence of Washington, then threatened by the Confederates under General Early. He pursued the latter and defeated him at Snickers Gap. In the ensuing campaign with the Army of the Shenandoah under Sheridan, he led his corps at Opequan and Fishers Hill, and was in command of the army at Cedar Creek, where he successfully rallied the army after its surprise. For gallantry at Spottsylvania he was brevetted Colonel, Brigadier-General at Cold Harbor, and Major-General, U. S. A., for the capture of Petersburg.

The following extract is quoted from the official order announcing his death to the Corps of Engineers:

"The thanks of the Legislature of his native State, Connecticut, were tendered to General Wright by resolution of June 14th, 1865, 'for his eminent services in the late war, as commander of the first brigade that set foot on the soil of South Carolina, in which were Terry and Hawley and Chatfield, and other noble soldiers of Connecticut; as commander of the First Division of the gallant Sixth Corps at Gettysburg, and through all the wearisome marches and sanguinary battles from that victory to the fall of the modest, yet great General Sedgwick; for the energy, skill and courage which from that sad day he displayed as commander of the Sixth Corps in many brilliant engagements, and especially at Cold Harbor, in front of Petersburg, at Washington during the last rebel raid, in the Shenandoah Valley at Opequan, Fishers Hill and Cedar Creek, and in the last bloody assault and terrible pursuit which, ending in the capture of Lee and his army, decided the fate of the Rebellion.'"

At the termination of the Civil War, in 1866, General Wright

returned to duty as a Lieutenant-Colonel in the Corps of Engineers. From this time until June 30th, 1879, when he was appointed Brigadier-General and Chief of Engineers, his most important service was as a member of the Board of Engineers for Fortifications and River and Harbor Improvements.

If the war period was a time that tried men's souls, the period directly following the war was a time that tried men's minds. The invention of high-power guns and the introduction of armored ships of war had revolutionized the systems of attack and defense. Old ideas had to be discarded, and it was necessary to work out the problems of military science with new data and from new points of view. At this time, also, the country adopted a policy of extensive river and harbor improvement, requiring the establishment of new methods of engineering administration and the investigation of new engineering problems, some of which were of great importance and perplexity. In connection with all these complicated and important questions, General Wright labored with patience, ability, wisdom and distinguished success. It is unnecessary to mention in detail all the important duties with which he was charged; the following Boards and Commissions, of which he was a member, are noted merely as examples of the character of his services:

Commission on East River Bridge; Commission to Europe to collect information upon the fabrication of iron for defensive purposes; Commission on the Sutro Tunnel; Board on Delaware Breakwater; Board on Heavy Ordnance; Board on Depressed Gun-Carriages; Board on Mouth of Mississippi River; Board on Paving Pennsylvania Avenue, Washington; Commission on Washington Monument, and Light-House Board.

General Wright was appointed Chief of Engineers on June 30th, 1879, and occupied that position until his retirement on March 6th, 1884. For this high and responsible office he was peculiarly well fitted, both by the extent and variety of his professional experience and by his personal character. By repeated service as assistant in the Engineer Bureau of the War Department he had gained full knowledge of the details of the work of that important office; and through his practical experience as a constructing engineer, and in connection with many boards on important engineering problems, he thoroughly understood the difficulties with which his officers had to contend, and they always received his earnest sympathy and encouragement.

He was a man of commanding presence and of great personal charm; quiet and gentle in disposition, yet wise, strong and decided in all the duties which required his action. He retired from the position of Chief of Engineers not only with the respect and admiration, but also with the affection of all the officers under his command.

General Wright was married, on August 11th, 1842, to Miss Louisa M. Bradford, of Culpeper County, Virginia. He is survived by two daughters, Mrs. Mary Wright Wootton, of New York City, and Mrs. Rosa Wright Smith, of Washington, D. C. After his retirement from active service he lived quietly at his residence in Washington, D. C., without engaging in professional work, and the charming hospitality of his happy home will long be remembered by those who were privileged to enjoy it. The memory of this noble, able and faithful soldier and gentleman is a rich legacy to his family and to the corps upon which he has conferred reputation and honor.

He was elected an Honorary Member of the American Society of Civil Engineers on March 3d, 1890.

EDGAR SHELDON CARY, M. Am. Soc. C. E.*

DIED JANUARY 5TH, 1883.

Edgar Sheldon Cary was a native of Fishkill Village, New York. In 1866, when he was twenty years of age, he was employed as chainman, rodman and leveler on the preliminary survey of the New York and Albany Railroad. The next year he went to the Albany and Susquehanna Railroad, where he served as draftsman at the general offices and as assistant on construction near Nineveh, New York. In the spring of 1868 he became leveler on the preliminary surveys of the Cherry Valley and Albany Railroad, and afterward became transitman on the location, and First Assistant Engineer on the construction, of the road. In the winter of 1868-69 he was a leveler on the preliminary surveys of the Boston, Hoosac Tunnel and Albany Railroad.

In September, 1870, he became Principal Assistant Engineer on the construction of the Lackawanna and Susquehanna Railroad, where he remained for about a year, being then transferred to the New York and Canada Railroad, and put in charge of a party revising location. Early in 1872 he was transferred to the New York and Albany Railroad as transitman on location, and in a few months was transferred back to his former position with the New York and Canada Railroad, where he remained until the completion of the work in June, 1873.

He then went to Albany, New York, and, after a short engagement under the City Surveyor, opened an office as engineer and surveyor. His chief work was a set of assessment maps of Bath and Little Falls, for which he made the surveys. During this time he was also in charge of a party on the location of the Boston, Hoosac Tunnel and Albany Railroad, and located the Albany and Greenbush Bridge.

In the fall of 1875 he became an assistant on the construction of the terminals of the New York Central and Hudson River Railroad, at Sixty-fifth Street and North River, New York City. In April, 1877, he became engineer for the contractor on the Riverside Avenue Improvement, in New York City, and about a year later joined the engineering corps engaged in the location and construction of the Second Avenue Elevated Railroad, in the same city. In October, 1879, he became engineer for the contractor on the construction of the Mad River Dam, near Waterbury, Connecticut.

In July, 1880, he resumed railway work, and was put in charge of a party making preliminary surveys and the final location of the Eastern Division of the Middletown Branch of the North River Railway. He was soon transferred to the New York, Ontario and Western

*Memoir prepared by the Secretary from information furnished by William H. Searles, M. Am. Soc. C. E., and from papers on file at the House of the Society.

Railway as Resident Engineer on construction, a position which he held at the time of his death. In his work on the Middletown Branch he was given only general instructions, but obtained very satisfactory results, both in alignment and ruling grade. The use of spiral curves was just then coming into use, and Mr. Cary proved himself a valuable assistant in the preparation of the formulas and tables used to facilitate laying out these curves in the field. The Middletown Branch was the first part of the line to receive spiral treatment, and Mr. Cary laid out the curves from the manuscript, the book not having been published.

While still in charge of the eastern half of the Branch, he made a survey of the river front of Newburgh which was remarkable for its accuracy and minuteness of detail. The map of this survey, drawn to a large scale, was used freely to project the several alternate lines proposed in the very difficult matter of getting a location for the West Shore Railroad through Newburgh, a distance of two miles, involving a tunnel of 400 ft., and an elevated causeway between retaining walls through many blocks, after a large number of valuable buildings had been removed. It was only after this removal that the center line, by no means a straight one, could be run. It confirmed perfectly the accuracy of the general survey.

Mr. Cary took an intense interest in all his work; indeed, he was an enthusiast in his profession, a rapid and industrious worker, quick to learn and seize upon any methods, new to him, by which better results might be achieved. He was a self-made man, and was ambitious to be still learning. Modest, genial and sincere, he commended himself to his superior officers, and won the esteem and respect of those under him. He never spared himself while anything remained to be done, and thus, while perfectly temperate in habit, he, through undue zeal, laid the foundations for the disease which carried him to an early grave. He left a wife and several interesting children to mourn his loss.

Mr. Cary was elected a Member of the American Society of Civil Engineers on November 1st, 1882.

ALFRED EPHRAIM HUNT, M. Am. Soc. C. E.*

DIED APRIL 26TH, 1899.

Alfred Ephraim Hunt was born in East Douglass, Massachusetts, March 31st, 1855, his parents being Leander Batchelor and Mary H. Hunt. On his father's side he belonged to the eighth generation descended from William Hunt, who in 1635 came from Salisbury, England, and became one of the original settlers of Concord, Massachusetts. Another ancestor served with distinction in Jackson's Additional Continental Regulars during the Revolution. Mr. Hunt's paternal grandfather was the founder of the works in Douglass, Massachusetts, where the celebrated Hunt's axes were manufactured. On his mother's side his ancestry is traceable directly back to Rev. Peter Thacher, at one time rector of St. Edmund's Church, Salisbury, England.

In the twenty-three years of his active professional life Mr. Hunt was not only prominent in several enterprises which marked epochs in the metal industries, but also took the part of a pioneer in the latest of these, the development of processes for the cheaper production of aluminum.

Mr. Hunt's common school education was obtained at the Roxbury High School, and he was graduated in 1876 from the Massachusetts Institute of Technology as a metallurgical and mining engineer. For several summers Mr. Hunt had obtained experience by service with the United States Coast Survey, and during a part of his senior year in the Institute he had engaged in analytical and metallurgical work for the Bay State Steel Company, of South Boston. Immediately after being graduated, he was engaged as Chemist and Assistant Manager of the open-hearth plant of these works, and here he assisted in the erection of the second open-hearth steel plant in the United States. At the suggestion of the manager of these works, Mr. Hunt was sent to Michigan to investigate the newly discovered iron ore deposits. His reports on the ore fields of Northern Michigan and Wisconsin had an important bearing on the development of ores in a region now famous for the extent and product of its iron mines. In 1877 he became Metallurgist of the Nashua Steel Company, Nashua, New Hampshire, and at these works he had charge of the reconstruction of the rolling mill and the building of a number of furnaces. In 1881 he removed to Pittsburg, Pennsylvania, and became Metallurgical Chemist and Superintendent of the Heavy Hammer Department of the Black Diamond Steel Works of Park Brothers and Company.

*Memoir prepared by the Secretary from a more extended memoir by John R. Freeman, M. Am. Soc. C. E., printed in *The Technology Review*, Vol. I, No. 3, and from papers on file at the House of the Society.

Mr. Hunt's widely known work in the testing and inspecting of engineering materials began in 1883, when, in connection with George H. Clapp, Assoc. M. Am. Soc. C. E., then of the firm of Park Brothers and Company, he started a metallurgical and chemical department of the Pittsburg Testing Laboratory, for the physical and chemical testing of materials and the inspection of iron and steel structures. In two years' time the business had attained such proportions that Mr. Hunt and Mr. Clapp acquired complete control of the Testing Laboratory and devoted their entire time to its affairs. Mr. Hunt had charge of the inspecting department of the business and of the inspection of the material for some of the most important metal structures erected in this country during the last fifteen years, including bridges over the Mississippi and Missouri Rivers and the Poughkeepsie Bridge over the Hudson.

Mr. Hunt's services were in frequent demand in consultation, in the courts and in the development of metallurgical processes. It was in this latter line of research that his attention was directed to the process for the reduction of aluminum, discovered by Mr. Charles H. Hall. This process had been experimented with at works where aluminum, then selling at $15 per pound, was produced, but had been abandoned in favor of the earlier processes. Mr. Hunt, however, was quick to recognize the merit of the scientific principles involved in the process, and in 1888 he organized the Pittsburg Reduction Company, raising the capital among his personal friends. Since its organization, Mr. Hunt had been President and General Manager of the company, and to his energy, judgment and enterprise the remarkable growth of the industry and the cheapening of the product to a selling price of 30 to 40 cents per pound are due.

Mr. Hunt's ancestors served in the Continental Army, and during his student days, he became warmly interested in the lectures on military science and tactics. He served in the Volunteer Militia of Massachusetts and New Hampshire during his residence in those States, rising to office; and on removal to Pittsburg he helped to organize the artillery arm of the State troops. For twelve years he served as the commander of Battery B of the National Guard of Pennsylvania, and had brought it to a high state of efficiency. On the outbreak of the war with Spain, the battery, to a man, enlisted within twenty-four hours of the President's call for troops, and Captain Hunt put aside his important business interests to lead his command. He was sent home from Chickamauga, an invalid, and, although he had not fully recovered, he determined to accompany his battery when it was ordered to Porto Rico. There he contracted malaria, which affected his heart, and he was discharged from the service, seriously ill, as the result of his exposure. He was on his way from Pittsburg to the seashore in search of rest and health, when, in a hotel in Philadelphia, he was seized with a hemorrhage which proved fatal.

Captain Hunt was prominent in masonic circles, being a Past Eminent Commander of Tancred Commandery No. 48, Knights Templar. He was also a member of the Syria Temple of the Shriners, and was a member of the Scottish Rite Masons in the Valley of Pittsburg. In this order he had attained the 32d degree.

He was married in 1878 to Miss Maria T. McQuesten, of Nashua, New Hampshire, and left one son, Roy Arthur Hunt.

Mr. Hunt was elected a member of the American Society of Civil Engineers on September 1st, 1886. He contributed in 1893 a paper on "A Proposed Method of Testing Structural Steel,"* which was presented at the International Engineering Congress, at Chicago, and for which the Norman Medal of the Society was awarded in 1894. He was also a Member of the Institution of Civil Engineers, the Iron and Steel Institute, the American Society of Mechanical Engineers and the American Institute of Mining Engineers. He was active in the movement for the purification of the water supply of Pittsburg, and at the time of his death was a member of a commission appointed by the city to investigate remedies for the smoke nuisance.

* *Transactions*, Am. Soc. C. E., Vol. xxx, p. 181.

WILLIAM BATEMAN LAWSON, M. Am. Soc. C. E.*

DIED JANUARY 27TH, 1901.

William Bateman Lawson was born in Chesterfield, Illinois, in 1846. Raised on a farm, he acquired sturdy habits of industry, and was possessed of a strong constitution and robust health. Upon the completion of his collegiate education at the Shurtleff College, Upper Alton, Illinois, in the year 1867, the natural inclination of his mind was toward scientific subjects, and it was very reasonable to expect that the profession of a civil engineer would be the most congenial to his nature.

The era of railroad construction was at its height throughout the West at this time, and Mr. Lawson commenced work as a rodman with a locating party on the Burlington and Missouri River Railroad, near Creston, Iowa, in May, 1868. Being of a studious nature and industrious, by careful attention to his duties, his advancement was rapid, and in 1870 he was in charge of construction work on the Burlington and Southwestern Railway. From this date until 1886 he was connected with various railroad enterprises and in responsible positions. In 1874 and 1875 he was engaged on the location and construction of the Bucksport and Bangor Railroad, in Maine. Afterward, he was in charge of construction on several branches of the Chicago, Burlington and Quincy Railroad in Illinois and Iowa. In 1881 and 1882 he was Chief Engineer of the Humeston and Shenandoah Railroad, in Iowa, from its inception until it was completed. In the latter part of 1883 he was appointed Chief Engineer of the Chicago, Burlington and Northern Railroad, and skillfully carried out the difficult task of locating the line along the eastern bank of the Mississippi River, where much engineering ability was displayed in overcoming the difficulties of crossing existing lines of railroad and in fitting the line to the somewhat peculiar conformation of river bluff and intervening valley lands. In the year 1886 the Chicago, Burlington and Quincy Railroad contemplated an extension of its system west from Denver, Colorado, through the Rocky Mountains, and Mr. Lawson was appointed Chief Engineer of the work, and personally made a thorough examination of the entire route between Denver and Salt Lake, and over much of the country between Salt Lake and the Sierra Nevada Range. He supervised the work of many engineer corps who made exhaustive surveys of the contemplated route, much of which was located and made ready for construction. On this work he showed a rare faculty for handling large enterprises to the best advantage, and was familiar with every detail of the entire work, scrutinizing and passing upon each problem as the work progressed. The construction of this extension was, how-

* Memoir prepared by H. A. Sumner, M. Am. Soc. C. E.

ever, deferred by the railroad company, and Mr. Lawson, who had in the meantime established for himself a home in Denver, opened an office as consulting engineer, and was engaged until the time of his death mainly on railroad and hydraulic work.

In 1893 he was engaged upon the difficult work of locating a portion of the Florence and Cripple Creek Railroad, a narrow-gauge road with 30° curves and 4% grades, and in 1899 he made a reconnaissance survey for the improvement of the Oregon Short Line in Idaho. His last work was the construction of a large reservoir system for the Colorado Fuel and Iron Company, near Pueblo, Colorado, his sudden death by apoplexy occurring before the entire completion of the enterprise.

Mr, Lawson was by nature peculiarly fitted for a civil engineer, having an investigating and critical mind, which not alone studied and mastered the theory, but which also tested the practicability of each subject investigated. He was a man of tireless energy and perseverance, and one of the most thoroughly posted engineers in the West, and his counsel was always wise and safe. He was an earnest friend to the younger members of the profession, and his assistance was given them in a quiet and unostentatious manner.

On February 10th, 1876, he was married to Miss Phebe A. Spofford, of Bucksport, Maine, who survives him.

He was elected a Member of the American Society Civil Engineers on February 1st, 1893, and was an active member of the Denver Society of Civil Engineers while that organization was in existence.

ROBERT IMLAY SLOAN, M. Am. Soc. C. E.*

DIED MARCH 3D, 1901.

Robert Imlay Sloan, son of William Henry and Caroline (Imlay) Sloan, was born February 23d, 1837, at Flemington, the county seat of Hunterdon County, New Jersey. His father was a prominent lawyer in Flemington, as also was his uncle, Peter I. Clark, whose name was well known throughout the State.

In 1852 Mr. Sloan commenced his professional career as Rodman with Captain John A. Partridge, Resident Engineer on the Flemington Branch of the Belvidere and Delaware Railroad. On this road Mr. Sloan was engaged until 1855, as Leveler, Transitman and Assistant Engineer, having charge of one section.

In the fall of 1855 he entered the Rensselaer Polytechnic Institute, of Troy, New York, and was graduated with the class of 1859. In 1866 he was Assistant Engineer on the St. Joseph and Topeka Railroad, and also on the St. Joseph and Marysville Railroad, both in Kansas.

From 1861 to 1864 Mr. Sloan was in the Quartermaster's Department of the Army of the Potomac, in Virginia. In 1865 he was surveyor of oil tracts in Venango County, Pa., and then Assistant Engineer on the Atlantic and Great Western Railroad, and on surveys for the improvement of Rock River and the Mississippi River, under the United States Government. He was City Engineer of Trenton, New Jersey, for four years.

On February 1st, 1876, Mr. Sloan was employed as Principal Assistant Engineer on surveys of the Metropolitan Elevated Railway, of New York City, consisting of the Gilbert (Sixth Avenue) and Second Avenue lines. Under William F. Shunk, Chief Engineer, he designed the foundations and track system of the Metropolitan Elevated Railroad, on the West Side, New York City. When the Sixth Avenue and Second Avenue lines were leased to the Manhattan Railway Company Mr. Sloan retained the same position in the latter company until August 1st, 1881, when he was appointed Chief Engineer. This position he held until March 1st, 1890, when he resigned, to become Chief Engineer of the Chicago South Side Elevated Railway during its construction. He remained with this road for some time after it was completed and in running order, and then, on account of failing health, he resigned and came East.

For two years he was interested in a lumber and planing mill, in connection with building houses by contract in New Jersey.

For a time he was an engineer for J. B. and J. M. Cornell, and

*Memoir prepared by the Secretary from papers on file at the Society House, and from information furnished by J. Waterhouse, M. Am. Soc. C. E.

looked after the erection of a number of buildings, among which was the Waldorf-Astoria Hotel in New York City.

At the time of his death he was a member of the firm Hall and Company of New York.

Mr. Sloan was an able and conscientious engineer, and stood high in the profession. He had a great many friends, and was well liked by all who became intimately acquainted with him. He was of a retiring disposition, and his naturally nervous temperament was accentuated by ill health. He would, at times, give vent to his feelings in a momentary flash of temper, when things did not go as he wished, but this would soon pass and he would become as calm and placid as though nothing had happened. He never harbored an ill feeling against anyone, and was kindly disposed to everyone he met.

Mr. Sloan had been brought up as a Presbyterian, and had an abiding faith in the future state, as laid down according to the Christian scheme, and would resent anything which seemed to cast doubt on such an idea.

On June 5th, 1878, Mr. Sloan was married to Miss Rupert, of Trenton, New Jersey, who, with one son, Robert I. Sloan, Jr., survives him.

Mr. Sloan became a Member of the American Society of Civil Engineers on October 1st, 1884.

HAMILTON SMITH, M. Am. Soc. C. E.*

DIED JULY 4TH, 1900.

Hamilton Smith was born near Louisville, Kentucky, on July 5th, 1840, and was the son of a distinguished lawyer, Hamilton Smith, Sr., originally of Durham, New Hampshire.

Until he was thirteen years of age, he attended the schools of Durham, but from 1854 to 1859 he studied engineering in his father's coal mines at Cannelton, Indiana, where he arose rapidly to be chief of the engineering and accountant's departments. From 1859 to 1867 he was engaged in developing collieries in Kentucky and Indiana, and in superintending the railroads and machinery connected therewith. When he left Cannelton in 1869, at the age of 29, to go to California, he was already known as a skilful mine manager and a leader of men.

His first work on the Pacific Coast was as Consulting Engineer of the Triumfo Mine, but his master work of that period was in hydraulic gold mining, while he was Engineer and Manager of the North Bloomfield and Milton Mines, in Nevada County. Here he designed and carried out large and important dams, pipe lines and other works, and became the recognized authority on all matters relating to hydraulic mining in California. The outcome of this and similar experience was his able and well-known work on "Hydraulics."

Mr. Smith was also connected with various other mines on the Pacific Coast and in Mexico, and was active in efforts to cheapen the manufacture of high explosives and in establishing the Vulcan Powder Works.

In 1881 he reported on the El Callao Mine, in Venezuela, for Messrs. Rothschild of London, and, as Consulting Engineer, he afterward designed and superintended the construction of the machinery and appliances for that mine.

In 1885 Mr. Smith established himself in London as a mining engineer, in partnership with the late Mr. Edmund de Crano, with whom, in 1886, he founded the "Exploration Company, Limited." This company was started with the modest capital of $100 000, but in 1889 its capital was increased to $1 500 000, and in 1896 it was again increased to $6 250 000.

As manager of that company he was actively interested in gold mining in South Africa, which he visited in 1892 and again in 1895, and took part in the formation of the Consolidated Deep Levels, The Transvaal and General Association, and other enterprises. The results of his investigations on the Rand, and those of Mr. H. C. Perkins, led to the formation of deep-level mining companies, the working of which

* Memoir prepared by the Secretary from information furnished principally by Clemens Herschel, M. Am. Soc. C. E.

was described by him at some length in an article in *The Times*, of January 17th, 1893.

Mr. Smith was also connected with the Alaska Treadwell, Alaska United, Alaska Mexican Gold Mining, Anaconda Copper Mining and many other first-class companies.

In 1895 his partner, Mr. de Crano, died, and Mr. Smith formed a partnership with his old friend, Mr. H. C. Perkins, the firm having their head office in New York City.

One of his last operations was to secure a controlling interest in the celebrated Mariposa Grant, of California.

Mr. Smith was married in 1886 to Mrs. Charles Congreve, *née* Miss Jennings, of New Orleans, Louisiana, who survives him.

He resided at Washington, D. C., and at Durham, New Hampshire, the home of his ancestors, and it was at the latter place that he died.

In 1890, in association with *Sir* Benjamin Baker and Mr. John G. Meiggs, he was active in the promotion of the Central London Railway, and in the engineering and financial features of this work he continued to be deeply interested. No small measure of the success of this undertaking was due to his untiring energy.

Mr. Smith contributed three papers to the *Transactions* of this Society, as follows: "The Flow of Water through Pipes";* "Water Power with High Pressures and Wrought Iron Water Pipe";† and "Temperature of Water at Various Depths in Lakes and Oceans."‡ He was awarded the Thomas Fitch Rowland Prize, for the second of these papers, in 1884.

Hamilton Smith was an able, energetic and successful engineer. He was elected a Member of the American Society of Civil Engineers on February 5th, 1879. He was also a Member of the American Institute of Mining Engineers, and of the Institution of Civil Engineers of Great Britain.

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* *Transactions*, Am. Soc. C. E., Vol. xii, p. 119.

† *Transactions*, Am. Soc. C. E., Vol. xiii, p. 15.

‡ *Transactions*, Am. Soc. C. E., Vol. xiii, p. 73.

WILLIAM STARLING, M. Am. Soc. C. E.*

DIED DECEMBER 11TH, 1900.

Although Mr. Starling frequently spoke of Kentucky as if it were his native State, he was born in Columbus, Ohio, on January 25th, 1839. His father was a Kentuckian, but, shortly after being married, moved to Columbus, where his uncle was a large landholder. The family did not remain in Columbus very long, however, but removed to New York, where William Starling attended the then well-known Clark's Grammar School, and later the University of New York, whence he was graduated in 1856, his standing being indicated by the fact that he delivered a philosophical oration.

In 1857, his father failed in business, and the family moved to the prairies of Illinois, where they remained three years, moving to Frankfort, Kentucky, at the end of that time. The Civil War was just commencing then, and William Starling entered the service of the United States Army, where he was soon made a First Lieutenant in the Third Kentucky Cavalry. Later, he was promoted to a captaincy in the Ninth Kentucky Infantry. He did his first engineering work during this military service, beginning with some work for which no regular engineer could be spared. He showed such natural ability and aptitude for the work that he was soon made Chief of Engineers on the staff of General Van Cleve, who commanded the Fifth Division of the Twenty-first Army Corps.

At the close of the war, he spent a short time in business in New York, after which he became interested in a plantation at Sunnyside, in Chicot County, Arkansas. He planted cotton from 1868 to 1882, selling his plantation in the latter year, when he moved to Greenville, Mississippi. Here he resumed his work as an engineer, being mostly engaged in levee work and United States Government surveys. He devoted much of his time and study to levee works and river hydraulics, in which fields he became a recognized authority, both in America and Europe, and was appointed to the position of Chief Engineer of the Mississippi Levee District, which position he held for about ten years.

After serving for a short time with the United States Engineers on work on the Mississippi River, he was appointed a Member of the Southwest Pass Commission, a body created for the purpose of formulating a project for securing a permanent deep channel through Southwest Pass, one of the mouths of the Mississippi River. This was his last, as well as his most important professional position, for he died of heart failure, at Greenville, Mississippi, shortly after the board had completed its work.

* Memoir prepared by Leon Elie Lion, Assoc. M. Am. Soc. C. E.

He wrote a number of technical papers, with most of which the members of this Society are familiar. His paper, "Some Notes on the Holland Dykes,*" is the principal, and won for him the Norman Medal. Besides this, he wrote several other papers, among which were, "On Flood Heights in the Mississippi River, with Especial Reference to the Reach between Helena and Vicksburg,"† "The Discharge of the Mississippi River,"‡ and "The Improvement of the Mississippi River."§ In addition, he contributed occasionally to the discussions on papers before the Society. Several papers, written for the technical press, are now published in pamphlet form under the title "The Floods of the Mississippi."

At Cornell University he delivered a series of lectures on hydraulics, which were highly spoken of. At the time of his death, he had practically completed a work on the Mississippi River, to which he had given many years of labor and study, and which had involved thousands of miles of travel. As yet, this work has not been given to the public.

Mr. Starling was a man of great force of character. He was daring in his undertakings, and manly and frank in his intercourse with men. His culture and breeding impressed all who came in contact with him. In manner he was cheerful, and was ever ready with anecdotes, which he could relate with the skill of a raconteur.

He had a disposition which it is fortunate for an engineer to possess: He was cheerful in adversity and among discomforts, a student in camp, admired and liked by his subordinates, respected by and influential with the men who employed him.

Mr. Starling was elected a Member of the American Society of Civil Engineers on September 7th, 1887.

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* *Transactions*, Am. Soc. C. E., Vol. xxvi, p. 559.

† *Transactions*, Am. Soc. C. E., Vol. xx, p. 195.

‡ *Transactions*, Am. Soc. C. E., Vol. xxxiv, p. 347; Vol. xxxv, p. 358.

§ *Transactions*, Am. Soc. C. E., Vol. xx, p. 85.

CHARLES HERBERT SWAN, M. Am. Soc. C. E.*

DIED APRIL 17th, 1899.

Charles Herbert Swan was born in Boston, Massachusetts, on August 17th, 1842. His parents were James G. and Matilda Loring Swan. He came from a long line of Revolutionary and other Massachusetts families, including the Lorings, Tufts, Dudleys and others.

He was educated in the Boston public schools, entered the Lawrence Scientific School of Harvard University in 1859 and was graduated thence in 1861. His school associates remember him as the same quiet, conscientious worker he continued to be all his lifetime; a good companion, musically disposed, a player on the flute.

In those days the construction of a million-gallon water plant was an event, and to take part in it a contribution to the practice of civil engineering in the United States. Such a piece of work was the construction of the Charlestown (Mass.) Water-Works by Slade, later by C. L. Stevenson, civil engineer, 1861-64; and Mr. Swan had a part in the whole of it. His name, with others, was, accordingly, duly recorded on the marble slab erected in the pumping station on the completion of those works, which, less than forty years later, are practically dismantled, and, with the memorial slab, run a fair chance, before long, of being broken up for old junk and building material.

His next work was on the Salem Water-Works, where he remained until the fall of 1869, during the last year, as Acting Chief Engineer. He then went to Providence, Rhode Island, as one of the assistant engineers to J. Herbert Shedd, M. Am. Soc. C. E., on water and sewerage works. He remained in Providence until 1881, except that he spent part of 1874 and 1875 in Europe on account of his health. In 1880 he had serious trouble with his eyes, and was obliged to discontinue work for three years. He removed to Boston in 1881.

In 1884 he went to Europe with Samuel M. Gray, M. Am. Soc. C. E., then City Engineer of Providence, to study the sewerage systems of various European cities, and prepared the historical part of the resulting report.

In 1886 he was employed by Rudolph Hering, M. Am. Soc. C. E., then Chief Engineer of the Chicago Water Supply and Drainage Commission, as a special assistant. Between the fall of 1887 and the spring of 1888 he was engaged in making a study, for the Water Supply and Sewerage Committee of the Massachusetts State Board

* Memoir prepared by the Secretary from information furnished by Clemens Herschel, M. Am. Soc. C. E., and from a memoir prepared by Howard A. Carson and Otis F. Clapp, Members, Am. Soc. C. E., for the Boston Society of Civil Engineers.

of Health, of the scheme of disposal of the sewage of the North Metropolitan Sewerage District by chemical precipitation.

He was teacher, for one term, at the Lawrence Scientific School, during the absence of Professor Chaplin, in the spring of 1889, giving instruction in the strength of materials, in hydraulics, and in water supply and sanitary engineering.

In 1889 he was appointed one of the assistant engineers on the Metropolitan Sewerage System, with which work he was more or less actively connected until the time of his death.

From October, 1894, to September, 1897, he was an assistant engineer on the Boston Subway, but was also employed on the Metropolitan Sewerage System during a portion of this period.

In the winter of 1897–98 he made a report on a projected joint system of sewerage for Salem and Peabody. From 1898 to 1899 he was again devoting his whole time to the Metropolitan Sewerage work, and had charge of the special hydraulic studies and the preparation of the text of the engineering portion of the report for the high-level gravity sewer for the relief of the Charles and Neponset River Valleys.*

On June 30th, 1870, he was married to Miss Carrie Cheney, a daughter of President O. B. Cheney, of Bates College, Lewiston, Maine. His widow and four sons survive him.

In its personal relations, Mr. Swan's life was most admirable. He was a loving husband and father. Ever since his twenty-first year he had been an active member of the Baptist Church, in Boston and in Providence, and at the time of his death he was the President of the legal society managing the property of the First Free Baptist Church of Boston.

It is characteristic of him that his funeral sermon should have been preached from this text which he had himself, though with great diffidence, only a short time before, suggested to the pastor as a text for a Sunday sermon, "with a few thoughts upon it," that he had jotted down:

"Blessed are the pure in heart: for they shall see God."

Mr. Swan was elected a Member of the American Society of Civil Engineers on March 16th, 1870. He became a member of the Boston Society of Civil Engineers in 1882.

* Mr. Swan's portrait, and a more extended biography, may be found in the *Journal of the Association of Engineering Societies*, for September, 1899, Vol. xxiii.

LEVIS PASSMORE PENNYPACKER, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 30TH, 1901.

Levis Passmore Pennypacker was the eldest son of Charles H. and Elizabeth A. Pennypacker. He was born in West Chester, Pennsylvania, on January 1st, 1864, and was educated at Friends' School; Taylor Academy, in Wilmington, Delaware; Swarthmore College; Adams Academy, Quincy, Massachusetts; and the Polytechnic Institute at Troy, New York.

When 16 years of age he entered the service of the Denver and Rio Grande Railroad Company, as a rodman, and spent some months in Colorado and Utah. In December, 1881, he entered the employ of the Pennsylvania Railroad and served as a rodman on the location and construction of the Philadelphia, Germantown and Chestnut Hill Railroad. In June, 1885, he was appointed Assistant Engineer of Construction with the Philadelphia Traction Company, and became Division Engineer in February, 1886. He was appointed Engineer in Charge of Construction on the Sansome Street Division in March, 1887, and Assistant Engineer of the United States Construction Company in April, 1887, and held the dual position until May, 1888, when he was transferred to Chicago, Illinois, as Assistant Engineer of the United States Construction Company and Assistant Engineer of the North Chicago Street Railway.

He took charge of the cable construction of the West Chicago Street Railway in November, 1888.

On August 10th, 1890, he was appointed Representative and Assistant Engineer to the Chief Engineer of the United States Construction Company, the Broadway Cable Construction Company and the Baltimore Traction Company.

In the same year he went to San Salvador, Central America, where he held a high position in the service of that republic as a civil and mechanical engineer. Becoming involved in a quarrel with Ezeta, President of San Salvador, he escaped from the country and returned to his home in West Chester.

* Memoir prepared by the Secretary, from papers on file at the Society House.

Soon afterward he received an invitation to enter the service of the President of Guatemala, as General Inspector and Superintendent of Construction of the public works then going forward in Guatemala.

Mr. Pennypacker, as a mathematician, electrician and draftsman, and also in field work, was skilful and energetic. He was very quick in his movements, and was endowed with great strength. He was thoroughly familiar with the country, and passed days and nights in the forests along the Matauga River, with only two negro assistants.

While on a business trip to San Lucia, a short distance west of Guatemala City, he was attacked by pneumonia, which proved fatal. He was buried at Escuintla with Masonic honors.

Mr. Pennypacker became an Associate Member of the American Society of Civil Engineers on April 1st, 1891. He was high in the Masonic Order, being a Knight Templar and a Shriner.

STEPHEN PASCHALL MORRIS TASKER, F. Am. Soc. C. E.*

DIED MARCH 19TH, 1900.

Stephen Paschall Morris Tasker was born in 1834, and was the son of the late Thomas T. Tasker, Sr., one of the founders of the firm of Morris, Tasker and Morris, pioneers in the manufacture of wrought-iron tubes in the United States.

He received his education in private schools, and, after serving an apprenticeship in the Pascal Iron Works, became a member of the firm in 1856–57, remaining in active connection therewith until after the removal of the works to New Castle, Delaware, and its incorporation and absorption by the National Tube Company.

He became a Fellow of the American Society of Civil Engineers on September 20th, 1872.

* Memoir prepared by Henry G. Morris, M. Am. Soc. C. E.